ENGINEERING NEWS-RECORD

JUL 5 1940

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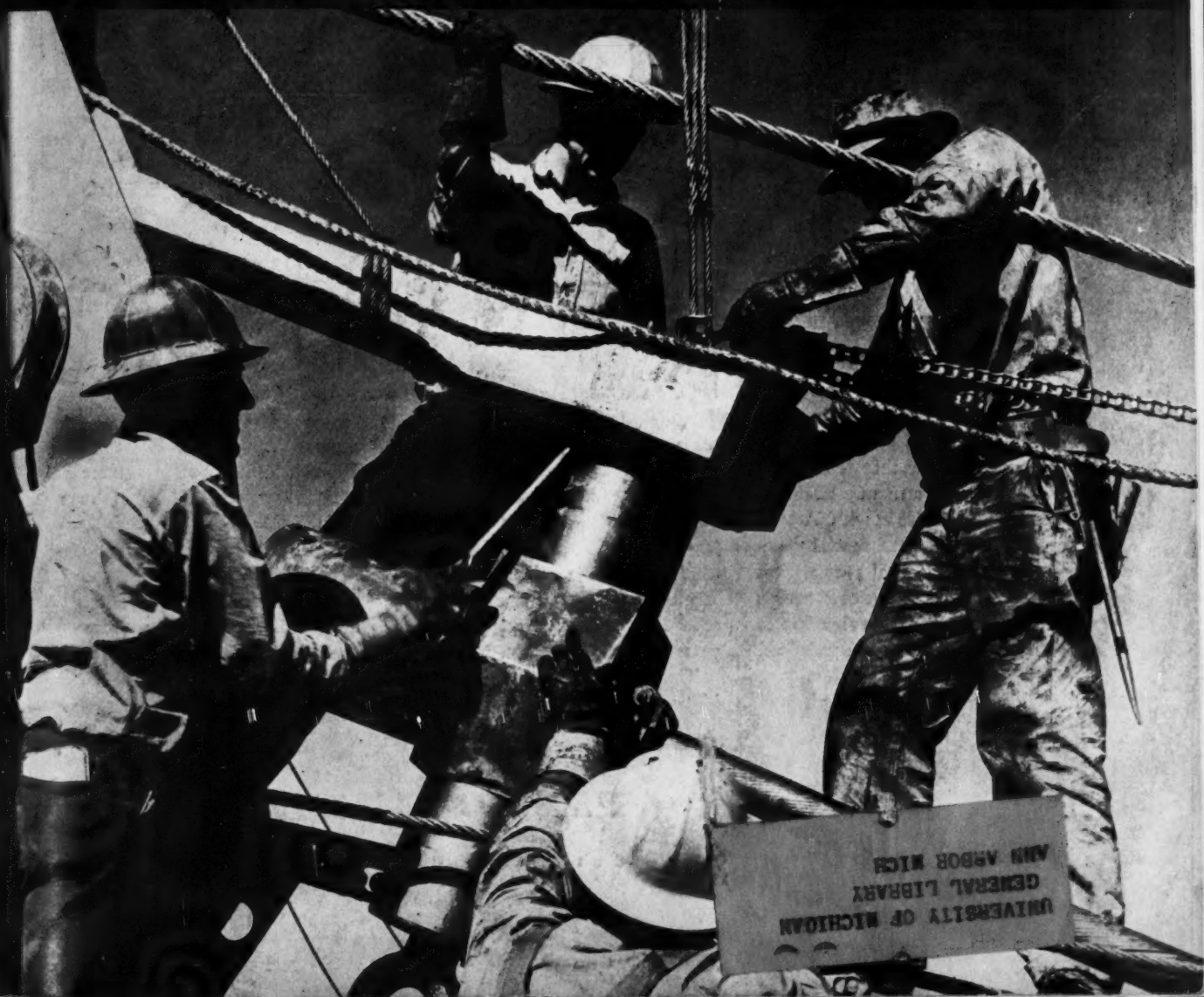
Basic Requirements for Airport Design—a Presentation of CAA Recommendations

Atop the headtower of Shasta Dam's cableway system a drawbar is connected to track cable trunnion.



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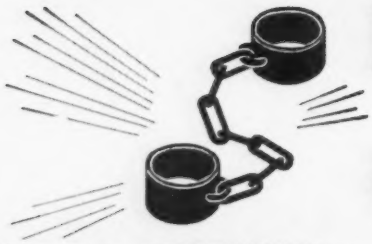
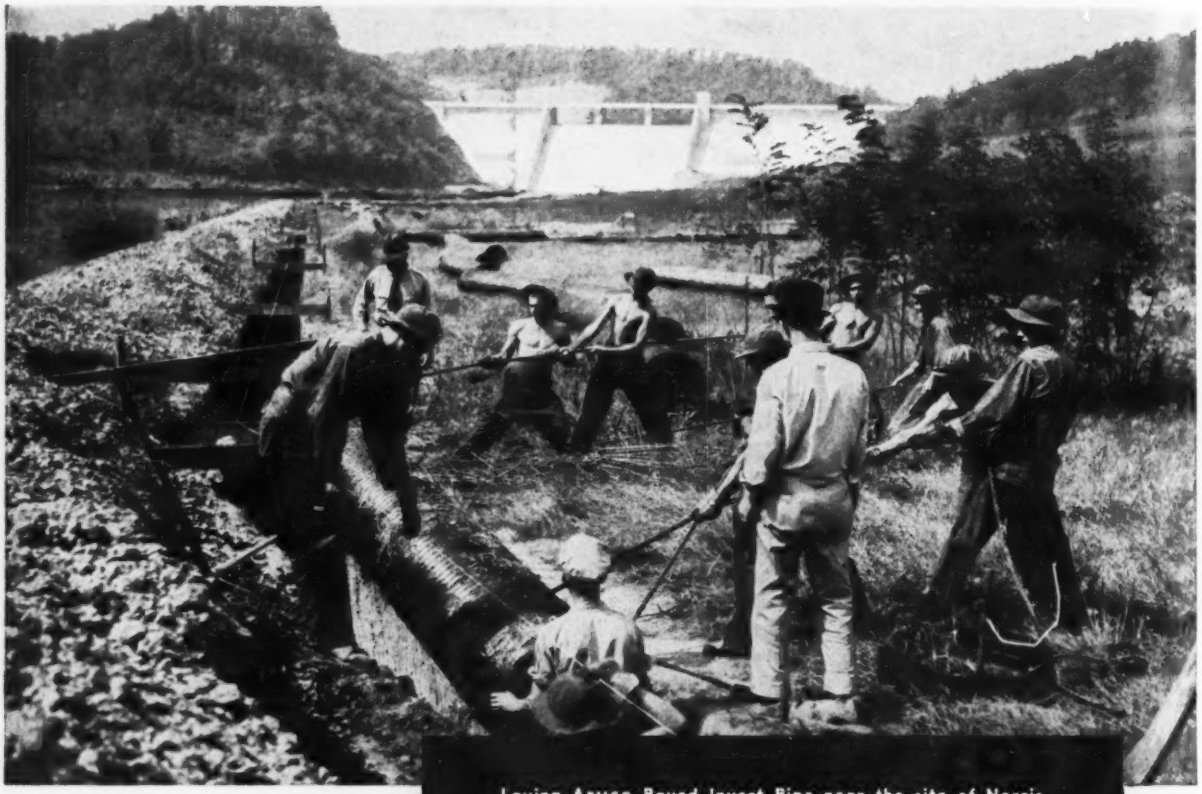


IT SHACKLES EROSION and Prolongs Pipe Life!



Laying ARMCO Paved Invert Pipe near the site of Norris Dam. Ordinarily the pipe is easily installed by considerably fewer men than the number available for this job.

• Old Man Erosion has been slowed down at last! ARMCO Engineers turned the trick by skilfully applying to the pipe a thick bituminous pavement to bear the brunt of "his" attack.

Inspections show that the bottom of *any* drainage structure normally wears out first. Now with ARMCO Paved Invert Pipe the special pavement protects this vital part against destructive scouring. The bottom lasts as long as the top, service life is doubled, and cost-per-year is less.

You'll find too that ARMCO Pipe has ample structural strength and material durability to meet every requirement. The flexible corrugated design makes it safe under load, impact or vibration. Sturdy band couplers assure a continuous, trouble-free conduit. And the galvanized ARMCO Ingot Iron base metal has demonstrated its enduring qualities in 34 years of corrosive service.

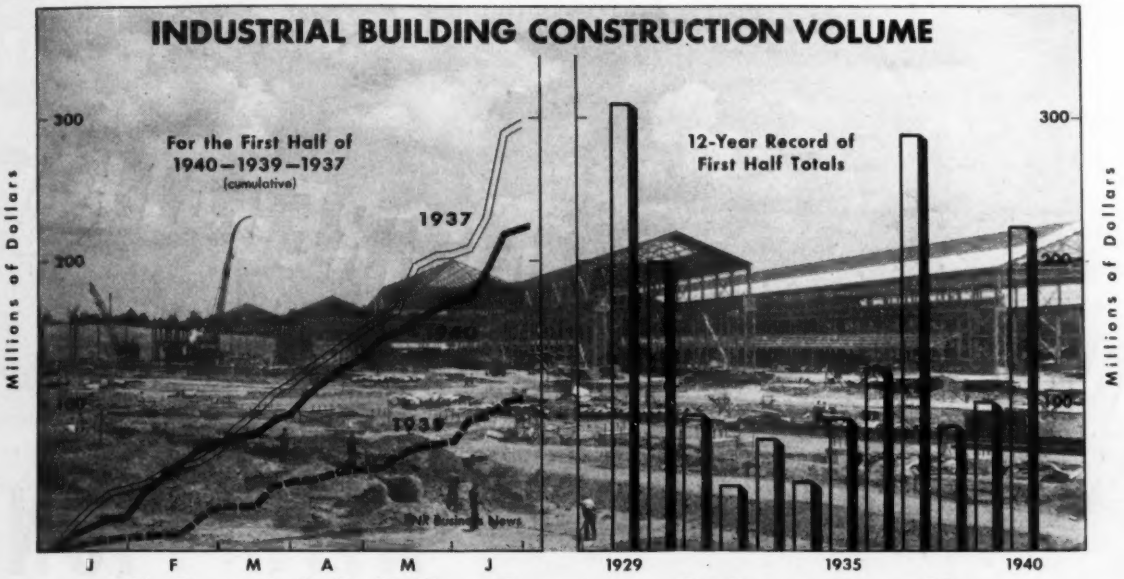
Thus you can readily see the possibilities for saving maintenance and replacement dollars with ARMCO Paved Pipe. Installation costs are likely to be lower too. With all its strength and durability ARMCO Pipe is light in weight for low-cost hauling and easy handling. Write us for information applied to your specific needs. Or better still, discuss your problem with the local ARMCO representative. Armco Drainage Products Association, 5037 Curtis St., Middletown, O.

ARMCO ARMCO **PAVED INVERT PIPE**

ORIGINATED BY ARMCO ENGINEERS

CONSTRUCTION TRENDS

Notes on significant movements in the construction industry



Industrial Building Reacts to Defense Spending

THE OPENING SALVO of defense spending jolted June industrial building construction to near-record levels. The month's volume, $51,130,000, has been exceeded only once in the past ten years, in June, 1937, when new steel mill construction in Pennsylvania swelled the volume to $87,565,000.

The Tennessee Powder Company's $15,000,000 powder plant was the largest single industrial project for the month directly attributable to war spending, but other industries no less important to defense also swung into action. Notable among the large June awards were: oil refining, aluminum reduction, plywood and glass factories, public utility plants, machine shops, and aircraft production facilities.

Industrial building awards averaged $12,783,000 per week for June, gaining 108 per cent over the average for the corresponding month last year, and topping the May, 1940, average by 63 per cent.

The June total brought industrial buildings to $222,400,000 for the first half of 1940, a figure exceeded only once in the eleven years since 1929. The 1940 six-month total more than doubled the volume for the first half of last year, and is two and one-half times greater than in 1938.

ENGINEERING NEWS-RECORD

Published every Thursday, Vol. 125, No. 1. Price 25¢ per copy. Allow at least ten days for change of address. All communications about subscriptions should be addressed to the Director of Circulation. 330 West 42nd Street, New York, N. Y. Subscription, domestic, Canada, Mexico, Central and South American countries $5.00 for one year, $8.00 for two years, $10.00 for three years. Great Britain and British Possessions 54 shillings for one year, 108 shillings for three years. All other countries $9.00 for one year, $18.00 for three years. Entered as second class matter October 29, 1936, at the Post Office at Albany, N. Y., under the act of March 3, 1879. Printed in U. S. A. Cable address "McGrawhill New York." Member A.B.P. Member A.B.C.

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Editorial and Executive Offices: 330 West 42d St., New York. Publication Office: 99-129 N. Broadway, Albany, N. Y. Branch offices: 520 N. Michigan Ave., Chicago; 68 Post St., San Francisco; Aldwych House, London, W. C. 2, England; Washington; Philadelphia; Cleveland; Detroit; St. Louis; Boston; Atlanta, Ga.

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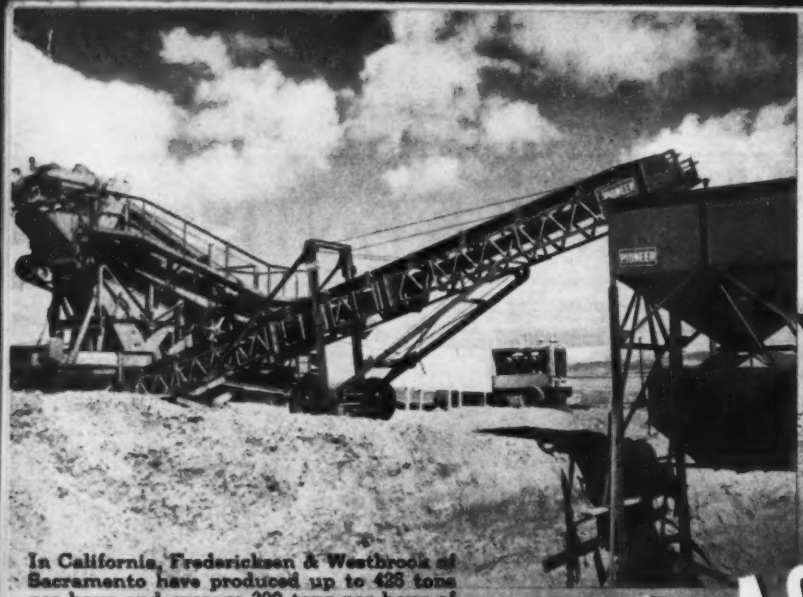
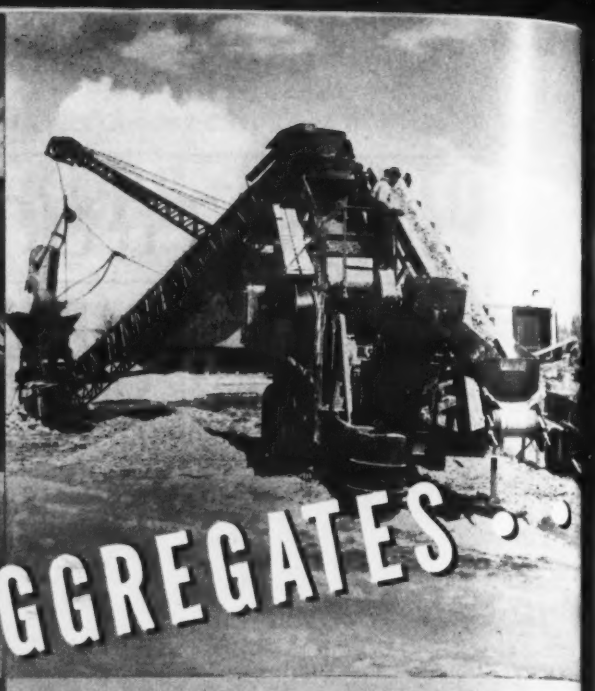
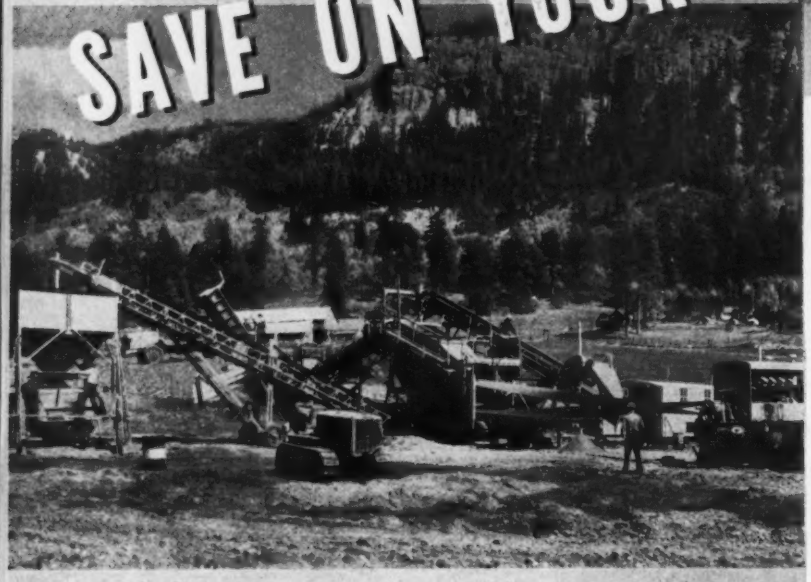


In California, Fredericksen & Westbrook of Sacramento have produced up to 425 tons per hour and average 300 tons per hour of gravel base course thru their Pioneer 48-V Plant.

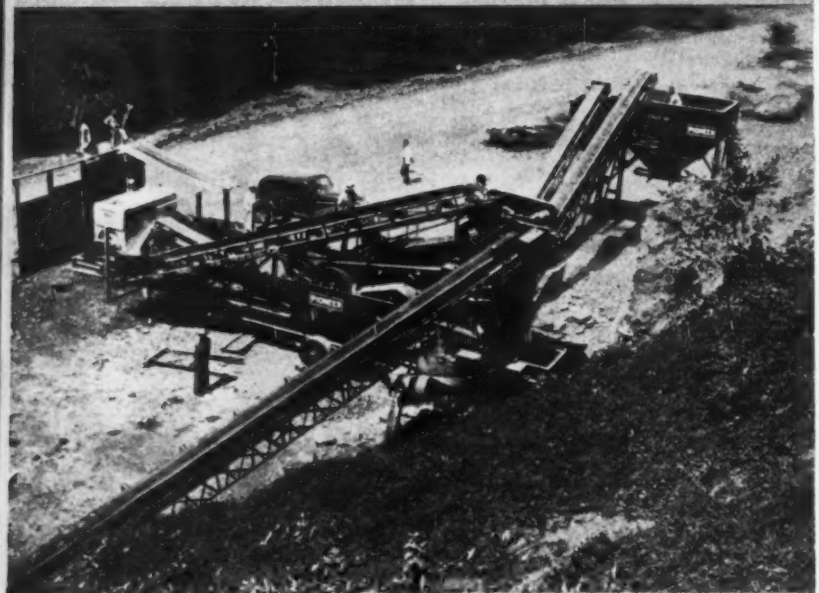
SAVE ON YOUR AGGREGATES . . .



In Michigan, Harry Pickitt produces more material at less cost per ton in his Pioneer 38-V Duplex Plant.



In Colorado, Larson Construction Company of Denver save by producing their aggregate in a Pioneer 48-V Duplex Plant.



In Colombia, South America, they produce base course, stone chips and reject excess fines in their Pioneer 34-V Duplex Plant.

WITH PIONEER

You, too, can save money by pro ducing your aggregates in a Pionee Crushing, Screening and Loadin Plant. Regardless where the job ma be—California, Michigan, Colorad or South America—Pioneer ca help you.

No matter how big or small yo job may be, there is a correct siz of Pioneer plant that will answe your requirements. In addition, th Pioneer plants will produce spec fication material, reject any desire percentage of fines and produc stone chips simultaneously.

If you are interested in saving o your aggregates, write to

PIONEER ENGINEERING WORKS, IN

MINNEAPOLIS, MINNESOTA

ENGINEERING NEWS-RECORD

Devoted to Buildings, Sewers, Waterworks, Bridges, Highways, Streets, Dams, Foundations, Earthwork, Flood Control, Public Works, Irrigation, Canals, Railroads

Publication Office: 99-129 North Broadway, Albany, N. Y. • Editorial and Executive Offices: 330 West 42nd Street, New York

F. E. SCHMITT AND WALDO G. BOWMAN, *Editors* • ALBERT E. PAXTON, *Manager*

Vol. 125 *July 4, 1940* No. 1

	PAGE
Contracts and Capital	27
The Week's Events	29
Obituaries	34
Comment and Discussion	36
Editorials	38
Basic Requirements for Airports	40
Aluminum Dust Combats Silicosis	45
Shasta Dam Construction Layout	46
Truss Bridge Becomes Long Cantilever HARRY J. ENGEL	50
Paving the Turnpike	53
Equipment Speeds Caissons and Piers F. L. COPELAND	58
Contractor-Engineer Relations	61
Mountain Water for Bethlehem	64
Lifting a River Over a Levee MAJOR JAMES D. ANDREWS, JR.	67
From Field and Office	71
Notes on Sewage Disposal	72
Men and Jobs	74
Contract Unit Prices	76
Materials and Labor Prices	78
New Aids to the Constructor	90
Construction Reports	97

Number of copies of this issue printed: 34,785

THIS WEEK

• At no time in the history of aviation in this country has the need for more and better airports been of such vital importance. The rapid obsolescence of existing airfields and the demands of a great military expansion program have focused attention upon our inadequate airport facilities for both commercial and military use. This week, *News-Record* presents Basic Requirements for Airport Design—a timely treatise setting forth the fundamentals of good planning, with special recommendations of the CAA. Every phase of design, planning and construction is covered.

• "Not a record for speed but a world's record for the lowest cost concrete placed according to specifications" was the objective in laying out the plant for mixing and placing of concrete on the great Shasta Dam project. How this plant was designed to deliver 5,700,000 cu. yd. of concrete over a radial cableway system to every point in the dam and powerhouse area is a story of construction at its best.

• Contractor-Engineer Relations of a Lifetime—a resume of experiences covering a third of a century in the construction business, as told by F. L. Copeland, vice-president of Bates & Rogers Construction Corp. This story will warm the heart of many an old-timer and inspire the younger men in the industry. Mr. Copeland has told his story with all the vitality of the pioneer.

THINGS TO COME

• Sabotage and Water Service—a special report to municipal authorities and waterworks officials, prepared by the editors of *Engineering News-Record*, will be published in the July 18 issue. Sabotage of water supply as a means of paralyzing industrial production is no myth. Therefore, this report on precautionary measures, obtained from a coast-to-coast canvass of key industrial areas, is of timely importance.

• Construction materials research and development as reported in papers at the convention of the American Society for Testing Materials will be a feature of the July 18 issue.

• The semi-annual index covering Vol. 124, January-June, 1940, inclusive, will be published in the July 18 issue.

Ryerson Reinforcing Steel-Service Helps Erect Big School* in Record Time

PROMPT "on schedule" delivery of 626 tons of Ryerson Reinforcing Steel was a big factor in completing this 595 ft. building in slightly less than 18 months. The builders of Pulaski High, along with thousands of other contractors, know by experience that Ryerson can be depended upon for prompt delivery of reinforcing steel, cut, formed, bundled and tagged according to exact specifications.

You can save time, trouble and money by taking advantage of Ryerson's *complete* service to Contractors and Builders—a service that includes formed bars, wire mesh, caisson rings, removable forms, and all accessories as well as every miscellaneous steel requirement from structurals to sheets, from corrosion-resisting steel to foundation bolts.

Our special Contractors and Builders Division will be glad to work with you on your next job. Whether your problem is one of intricacy of design, large or small tonnage, special service, or limited time, a Ryerson engineer can probably save you time and money. On your next job get in touch with the plant nearest you.

JOSEPH T. RYERSON & SON, INC. Plants at: Chicago, Milwaukee, St. Louis, Cincinnati, Detroit, Cleveland, Boston, Buffalo, Philadelphia, Jersey City.

*Pulaski High School, Largest in Milwaukee.
Contractor: Kroening Engineering Corporation, Milwaukee.

Bars are quickly cut to size.

. . . bent and formed to specifications.

. . . bundled and tagged ready for the forms.

RYERSON STEEL-SERVICE

CONTRACTS AND CAPITAL

ENR CONSTRUCTION VOLUME AND NEW CAPITAL . . . FHA MORTGAGES

(,000 omitted)	June 1939 (5 wk.)	June 1940 (4 wk.)	% Change†	Six Months 1939 (26 wk.)	Six Months 1940 (26 wk.)	% Change
Total Construction	$268,395	$252,763	+18.0	$1,563,224	$1,389,616	−11.0
Private Construction	60,509	92,044	+90.0	390,148	484,244	+24.0
Public Construction	207,886	160,719	−3.0	1,173,076	905,372	−23.0
Federal	47,136	49,834	+32.0	150,419	215,431	+43.0
Total New Productive Capital	$116,632	$51,132	−45.0	$951,186	$632,535	−33.0
Private Investment	50,782	41,133	+1.0	447,162	538,382	+20.0
Federal (non-federal work)	65,850	9,999	−81.0	192,597	94,153	−51.0
Federal (federal work)	.....	.....	.....	311,427	.....	.....
FHA Mortgages Selected for Appraisal	$101,501	$108,446*	+7.0	$602,105	$634,733*	+5.0

†Adjusted for difference in number of weeks. *Preliminary.

CONSTRUCTION COSTS . . . WAGE RATES . . . PRICES

	July 1939	July 1940	% Change	Change June to July 1939 June	1939 July	1939 %	Change June to July 1940 June	1940 July	1940 %
ENR Construction Cost Index, 1913 = 100	234.94	242.15	+3.1	235.02	234.94	‡	241.59	242.15	0.2
ENR Building Cost Index, 1913 = 100	196.78	201.68	+2.5	196.41	196.78	+0.2	201.65	201.68	‡
ENR 20-CITIES' AVERAGE									
Common Labor	$0.684	$0.707	+3.4	$0.684	$0.684	0	$0.703	$0.707	+0.6
Skilled Labor (Av. 3 trades)	1.444	1.476	+2.2	1.436	1.444	+0.6	1.472	1.476	+0.3
Bricklayers	1.538	1.568	+2.0	1.514	1.538	+1.6	1.568	1.568	0
Structural Ironworkers	1.499	1.535	+2.4	1.499	1.499	0	1.520	1.535	+1.0
Carpenters	1.296	1.330	+2.6	1.296	1.296	0	1.328	1.330	+0.2
Cement, per bbl	$2.60	$2.51	+3.5	$2.60	$2.60	0	$2.51	$2.51	0
Reinforcing steel, cwt	2.45	2.43	−0.8	2.45	2.45	0	2.45	2.43	−0.8
Struct. steel, base price	2.10	2.10	0	2.10	2.10	0	2.10	2.10	0
Sand, per ton	1.21	1.17	−3.3	1.23	1.21	−1.6	1.18	1.17	−0.8
Lumber, 2x4 Fir, per M ft.	32.38	34.88	+7.8	32.22	32.38	+0.5	34.88	34.88	0
Lumber, 2x4 Pine, per M ft.	28.86	31.15	+7.9	29.15	28.86	−1.0	31.59	31.15	−1.4
Brick, common, per M	14.24	14.34	+0.7	14.26	14.24	−0.1	14.32	14.34	+0.1
Ready-mixed concrete, c.y.	7.09	6.94	−2.1	7.11	7.09	−0.3	6.98	6.94	−0.6
Struct. clay tile, 3x12x12	74.27	74.49	+0.3	74.27	74.27	0	74.49	74.49	0
Paving asphalt, cnrs, ton	13.94	14.16	+1.6	13.94	13.94	0	14.19	14.16	−0.2

‡Less than .05 per cent change.

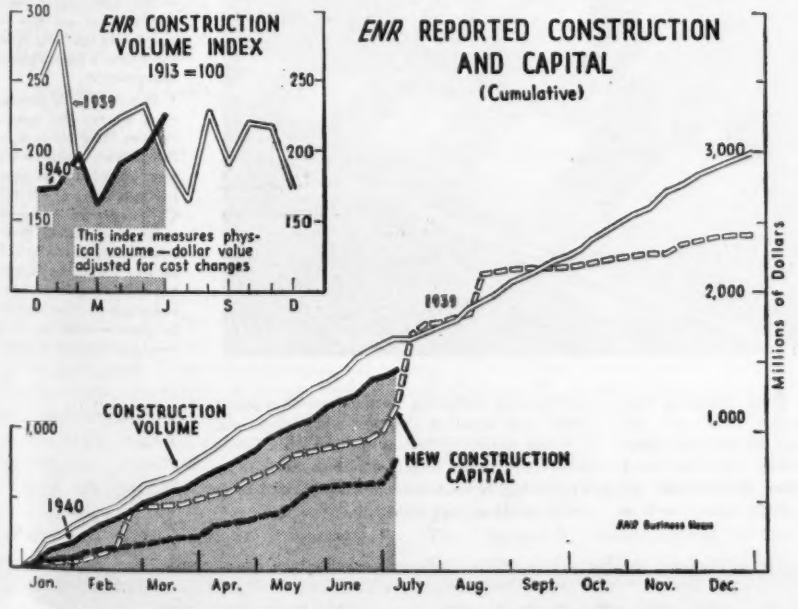
MATERIAL SHIPMENTS . . . BUILDING PERMITS

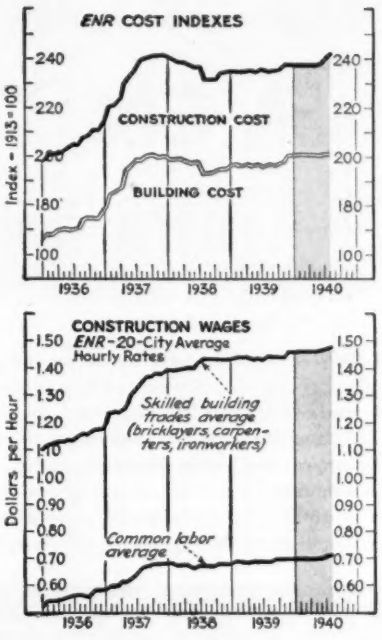
	June 1939	June 1940	% Change	May 1940	% Change May to June
Lumber (% 1929 wk. seas. av.) N. L. M. A.	69.3	73.3	+5.7	74.0	−1.0
Steel (% operating capacity) A. I. S. I.	54.2	84.8	+56.4	71.4	+18.8

	May 1939	May 1940	% Change	Five Months 1939	Five Months 1940	% Change
Fabricated Struct. Steel, tons, A. I. S. C.	125,818	109,191	−13.2	540,713	529,051	−2.2
Cement, thous. bbl., U. S. B. of M.	12,748	13,241	+4.0	41,533	40,579	−2.0
Building Permits, Dun & Bradstreet (000 omitted)	$111,489	$126,341	+13.3	$500,458	$534,678	+6.8

RENT INDEX . . . EMPLOYMENT

	May 1939	May 1940	% Change	Apr. 1940	% Change Apr. to May
Rent (Housing) Index, N. I. C. B.	86.2	86.7	+0.6	86.7	0
Est. Total Employment, N. I. C. B.	44,924,000	46,377,000	+2.3	45,977,000	+1.0
Est. Constr. Employment, N. I. C. B.	2,077,000	1,953,000	−6.0	1,934,000	+1.0





CONSTRUCTION WAGES
ENR-20-City Average Hourly Rates
Dollars per Hour
Skilled building trades average (bricklayers, carpenters, ironworkers)
Common labor average
1936 1937 1938 1939 1940

CONTRACTS

(Thousands of dollars)

	Week Ending July 6 1939	June 27 1940	July 4 1940
Federal	$8,070	$4,752	$2,588
State & Municipal	22,007	18,696	21,118
Total public	$30,077	$23,448	$23,706
Total private	7,472	11,421	7,893
TOTALS	$37,549	$34,869	$31,599
Cumulative 1940 (27 weeks)			$1,421,215
1939 (27 weeks)			$1,600,773

Note: Minimum size projects included are: Waterworks and waterways projects, $15,000; other public works, $25,000; industrial building, $40,000; other buildings, $150,000.

NEW PRODUCTIVE CAPITAL

	Cumulative 1939 27 Wks.	1940 27 Wks.
NON-FEDERAL	$787,449	$831,956
Corp. Securities	130,755	92,225
State & Mun	329,097	461,542
R.E.A. loans	43,732	.....
U.S.H.A. loans	146,419	87,204
R.F.C. loans	2,446	6,985
Fed. Aid-Hwy	135,000	184,000
FEDERAL	$311,427	$7,000
TOTAL CAPITAL	$1,098,876	$838,956

FHA MORTGAGES

	Week Ending July 1 1939	June 22 1940	June 29 1940
Selected for appraisal	$22,071	$26,037	$26,500*
Cumulative 1940 (26 weeks)			$634,773*
1939 (26 weeks)			$601,353

* Subject to revision.

ENR INDEX NUMBERS

Base = 100	Construction Cost 1913	1926	Building Cost '13	'26	Volume '13	'26
July, 1940	242.15	116.40	201.68	109.02	...	...
June, 1940	241.59	116.13	201.65	109.00	225	99
May, 1940	238.86	114.82	201.17	108.75	201	88
July, 1939	234.94	112.94	196.78	106.37	166	73
June, 1939	235.02	112.97	196.41	106.17	193	84
1939 (Av.)	235.51	113.21	197.44	106.73	211	92
1938 (Av.)	235.86	113.34	196.83	106.40	197	86
1937 (Av.)	234.71	112.66	196.15	106.07	172	75
1936 (Av.)	206.42	99.15	172.18	93.14	185	81

June Construction Table on page 34

NO MORE CURB BLACKOUTS EVEN ON RAINY NIGHTS

with new White Reflecting Curb

THE DARKER the night and the wetter the pavement, the greater is the driving visibility on roads guarded by White Reflecting Curb made with Atlas White cement. Under rain, the visibility of smooth concrete curb and pavement drops to practically zero. But on main New Jersey highways today, a new type curb with white concrete face and reflecting saw-tooth design shows up even brighter on rainy nights than in clear weather—*under no other illumination than ordinary car headlights.*

Moderate in first cost, less expensive in upkeep, and as permanent as the road itself, this simple but scientific contribution to traffic safety pays for itself in short time.

Atlas White cement is used for White Reflecting Curb because white reflects the maximum amount of light and gives greatest contrast to surrounding pavement by day and night. For further information, write Universal Atlas Cement Co. (United States Steel Corporation Subsidiary), Dept. C3, Chrysler Bldg., N. Y. C.

All photos by New Jersey State Highway Department

FIG. 1 Smooth White Curb—Dry and Wet

Fig. 1. Here is a strip of ordinary smooth white concrete curb under car headlights at night. It's only slightly more visible than the road. And when water (shown by arrow) covers the curb and road, there's a complete blackout. The curb disappears.

Here's the reason: Smooth curb and road reflect most headlight rays forward, away from the driver; visibility is poor. On a rainy night, water fills the slight irregularities of smooth curb and pavement, causes them to act even more like a mirror, and to reflect even more headlight rays forward, away from the driver; visibility is nil (shown by arrow). Road and curb are almost completely obscured.

FIG. 2 White Reflecting Curb on rainy night

Fig. 2. White Reflecting Curb makes a gleaming ribbon of light extending far in front of the car. Even under unfavorable conditions—on a rainy night, as here—the curb itself seems to be lighted. But actually this is simply due to the light reflected from the driver's headlights—no other illumination.

The reason is simple: Smooth curb wastes the headlight rays, reflects them forward, away from the driver; visibility is poor. But the specially designed saw-toothed faces of White Reflecting Curb conserve the headlight rays, catch and reflect them back to the driver; visibility is high. And on wet nights the film of water on the reflecting faces intensifies the reflection—illumination is even brighter—visibility is even higher—night driving is safer.

FIG. 3

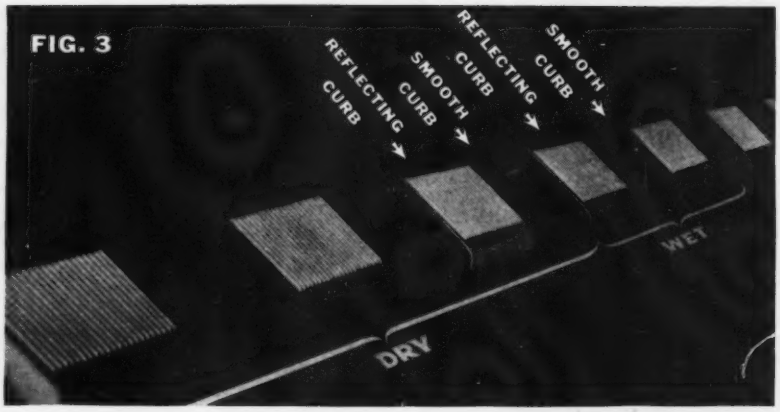


Fig. 3. A New Jersey State Highway Test: Alternate sections of smooth white curb and reflecting white curb under dry and wet conditions as seen by night drivers, under car headlights only, with no other illumination. All sections, the smooth ones which appear dark, as well as the reflecting sections which appear white, are made of identical white materials.

White Reflecting Curb made with Atlas White cement provides a curb that defines the road by day, is highly visible on dry nights, and even more visible on wet nights.

WHITE REFLECTING CURB

MADE WITH ATLAS WHITE CEMENT

Specification for Five Cements Adopted by A.S.T.M.

Multiple standard accepted with general approval. Much specification work and papers cover a wide range of materials. W. M. Barr elected president

Adoption of a portland cement specification covering five different types of cement featured the annual meeting of the American Society for Testing Materials at Atlantic City last week. The action was the first fundamental departure from the single cement standard maintained for 31 years, since original adoption of a portland cement specification by the society. The five types are: normal, moderate-heat, high-early-strength, low-heat and sulphate-resisting cement.

Aside from this action and the added emphasis given to cement problems by P. H. Bates in the Edgar Marburg lecture, which was a challenge to more intensive study of cement in both production and service, metals were the leading theme of the meeting. In contrast to some 30 papers on metals, there were only ten on cements, concrete, brick and mortar and a baker's dozen on bitumens, woods, plastics and miscellaneous materials. Several important specialist symposiums discussed radio-graphic testing, spectroscopy, water for industrial use, and chemical analysis.

W. M. Barr, New A.S.T.M. Head

Standards in national service

Speaking on "Material Standards in National Preparedness," Lieut.-Col. William C. Young of the War Department told the society that standard specifications are necessary for adequate military procurement and preparation. They are especially important because the many specialized requirements that have been developed by the military authorities during recent years must be adapted to the maximum range and rapidity of supply, and specifications must therefore be brought as closely as possible in accord with industrial conditions and commercial standards. The society pledged all possible assistance to the government and the supply industries in connection with national materials problems.

Retiring President H. H. Morgan in his presidential address reviewed the evolution of standards and stressed their importance to commercial intercourse and the development of industry. "Standardization is the bedrock of civilization," he said, pointing to the early establishment of standard weights and measures and the like. The advance of standards has been continuously in the direction of distinguishing between significant and measurable qualities from the insignificant and intangible, and progress will continue in the same direction, he asserted.

Honorary membership was conferred on Cloyd M. Chapman, consulting engineer; H. E. Smith, former engineer of tests of the New York Central system; and Thomas R. Lawson, former professor of civil engineering at Rensselaer Polytechnic Institute. Certificates were awarded to individuals and firms who have held membership in the society for forty years.

The Charles B. Dudley medal for the best original contribution on research in engineering materials during the past year was awarded to T. F. Willis and M. E. DeReus, of the Missouri Highway Department, for a paper on "Thermal Volume Change and Elasticity of Aggregates and Their Effect on Concrete," which also won the authors the Sanford E. Thompson award.

W. M. Barr, chief chemist of the Union Pacific Railroad, Omaha, was elected president of the society for the coming year. H. J. Ball becomes vice-president.

Committee work

Extensive standardization activity coupled with the study of material properties and service was reported in all departments of the society's work. Much of the development of specifications was refinement and revision, but also an exceptionally large number of new specifications was reported. The session brought the total of the society's standard and tentative specifications to 950, as compared with 885 a year ago.

Some major items of committee work in the field of civil engineering construction materials are noted below.

Masonry Units—No important speci-

(Continued on p. 32)

• • •

Battery-Brooklyn tunnel loan assured

Mayor La Guardia of New York City has been assured by Jesse H. Jones, Federal Loan Administrator, that the Reconstruction Finance Corporation will loan $57,000,000 for construction of the $80,000,000 vehicular tunnel to join the Battery on the lower tip of Manhattan to the Borough of Brooklyn. To save interest, the money will not be loaned as a lump sum but as needed.

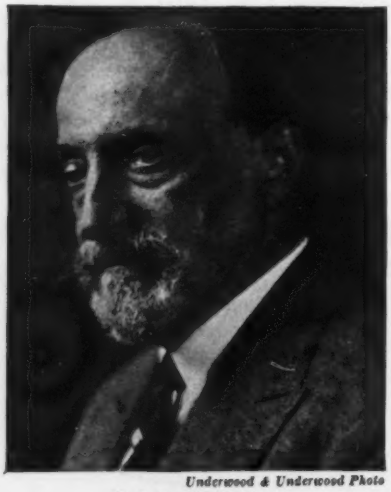
• • •

Gas firm seeks permit for 877-mile pipeline

The Independent Natural Gas Co., a subsidiary of the Phillips Petroleum Co., with main offices at Bartlesville, Okla., has filed application with the Federal Power Commission for a certificate to permit construction of a natural gas pipe line from Sherman County in the Texas Panhandle to Milwaukee, Wis., a distance of about 877 miles. Estimated initial construction is given as about $28,700,000.

Ralph Modjeski dies; bridge engineer

Ralph Modjeski, 79, leader of the American bridge engineering profession for many years and engineer or consultant for many of our largest bridges, died June 26 in Los Angeles after a long illness. His most famous works include the Quebec cantilever, the Philadelphia-Camden suspension bridge and the San Francisco Bay Bridge. His early fame, however, was built on the construction of a series of notable simple span railroad bridges across the Mississippi, including the Thebes and Memphis bridges.



Underwood & Underwood Photo

Ralph Modjeski

Born in Poland, he had come to this country in 1876 but was educated in Paris (Ecole des Ponts et Chaussees) before joining George S. Morison in 1885 to work on the construction of the Union Pacific Bridge over the Missouri at Omaha. Soon afterwards he designed the first bridge at Memphis, Tenn., across the Mississippi River. In 1893 he opened consulting offices in Chicago.

Early in his consulting practice Modjeski served as chief engineer of construction of the combined highway and railway bridge across the Mississippi at Rock Island, Ill. Soon afterward he constructed another Mississippi bridge, that at Thebes, Ill., below Rock Island. Other structures for whose planning or construction he was responsible during his earlier career include the Northern Pacific Bridge over the Missouri at Bismarck, N. D.; bridges over the Columbia and Willamette rivers at Portland, Ore., for the S. P. & S. Ry.; the McKinley Bridge across the Mississippi at St. Louis; the Celilo Bridge over the Columbia River; the Broadway Bridge over the Willamette River for the city of Portland, Ore.; the Harahan Bridge over the Mississippi River at Memphis; and the Metropolis Bridge over the Ohio River at Metropolis, Ill. Reconstruction of the Poughkeepsie railroad bridge and design and construction of the Mid-Hudson (highway) Bridge at Poughkeepsie were also under his direction.

At the time the stability of the Manhattan Bridge anchorage was in question Modjeski was head of a consulting board that confirmed the safety of the structure. Soon afterward he was selected as member and later chairman of the engineering board in charge of building the Quebec cantilever bridge across the St. Lawrence, in place of the originally planned structure that failed by collapse of a compression chord in 1907. He was also head of the board of engineers that designed and directed the construction of the Philadelphia-Camden Bridge over the Delaware. Through many years he studied the problem of a San Francisco Bay crossing, and was a member of various engineering boards that advised the authorities on this problem, finally becoming head of the board that designed and built the bridge now in service. Other structures for which he was designer or consultant include the Ambassador Bridge over the Detroit River, the Tacony-Palmyra Bridge over the Delaware, and the Huey Long Bridge over the Mississippi at New Orleans.

In his earlier consulting practice Ralph Modjeski was associated with Alfred Noble and later for a number of years with Daniel E. Moran. Still later his partners included Frank Masters, Clement E. Chase and Montgomery Case.

JOBS OF THE WEEK

ARKABUTLA DAM, Mississippi.

Forcum James Construction Co., Dyersburg, Tenn.; Pioneer Construction Co.; and H. N. Rodgers & Sons Co., Memphis, will build Arkabutla Dam on Coldwater River about 7 mi. northeast of Prichard, Miss. for U. S. Engineers, Vicksburg, Miss. This is part of the Yazoo River flood control work. $5,158,157.

SHIPWAYS, San Francisco, Calif.

Bethlehem Steel Co. (Shipbuilding Division), constructing 2 shipways by own forces, at San Francisco plant. $500,000. Navy Department will advance money for this work as well as $3,175,000 for company's plant at Alameda.

SHOP and OFFICERS QUARTERS, Jacksonville, Fla.

Doyle & Russell, Richmond, Va., low bidders on superstructure, assembly and repair shop, $763,330; T. A. Loving & Co., Goldsboro, N. C. for bachelor officers quarters and administration building, $764,600, for Naval Airbase.

WATER PIPELINE, Hartford, Conn.

Lock Joint Pipe Co., Ampere, N. J., low bidder for constructing Barkhamsted-Napaug and Collinsville bypass pipelines in towns of Barkhamsted, New Hartford, Canton and Avon, $939,306. Board of Contract & Supply, Metropolitan District, Richard Dillon, clerk, Municipal Bldg., Hartford, Conn. took bids. Caleb Saville, engineer.

INCINERATOR, Atlanta, Ga.

Greenawalt Engineering Co., New York, N. Y., low bidder constructing incinerator for City of Atlanta, $648,900. This plant, with a capacity of 500 tons daily, will be largest incinerator in the South, and one of the few in this country utilizing waste heat for steam generation. As in former operations at an old plant, the steam will be used to heat downtown office buildings. Robert & Co., Atlanta, engineer.

FORT SUPPLY RESERVOIR, Fort Supply, Okla.

Leo Sanders, Oklahoma City, Okla. was low bidder for closure section of earthen dam, spillway, riprap and bank protection for Fort Supply Reservoir, $2,920,666.

RAILROAD BRIDGES, Connecticut.

George F. Collins Co., New York; Silliman & Godfrey Co., Bridgeport; and Leake & Nelson Co., Inc., Bridgeport, awarded contract for constructing 16 bridges, 12 ft. wide by 36 ft. long, between Connecticut state line and New Haven for New York, New Haven & Hartford R.R. $1,250,000.

NOTE—Additional bidding and contract news on over 600 projects, large and small, appears in the Construction News Section beginning on page 97.

USHA gets Power to Meet Housing Needs of Defense Program

New law does away with need for local funds and restrictions on tenant income for projects favored by President

The U. S. Housing Authority can take over the major burden of meeting the housing needs created by the defense program under authority of legislation passed last week. Essentially, the new law exempts from the requirements of local contribution and from the restrictions on tenant income, USHA projects which are declared by the President to be needed for the national defense.

Such projects fall into two classes—those providing housing around army and navy facilities for enlisted men with families and for civilian employees; and those providing housing for workers in new or expanded armament factories.

As regards the first class, the new law provides that projects be built at the request of the Army or Navy. They would be built either by these services or by USHA, would be financed and owned by USHA and would be operated by the military services.

New housing around defense plants would be built either by USHA itself or by a local housing authority, but in either case, there would be no local contribution. Only restriction on rentals are that they be within the reach of those for whom the project is intended.

To carry out this work, USHA has unused authority to borrow $180,000,000. This money has not previously been borrowed because there is no subsidy money available for projects which might be built with it; this question would not arise on the defense projects. How the loans would be repaid is not made clear in the legislation. Presumably rentals would be somewhat higher than on the present subsidized housing and might meet most of the cost. It is understood that additional legislation on the subject is being prepared by the defense commission.

There is little hope in Washington that much of the needed housing will be privately provided. The discouraging experience of FHA with its attempts to stimulate construction of rental housing at comparatively low rents seems to indicate that even in normal times builders do not regard housing for workers as an attractive proposition, and the uncertainties of the defense program make the situation even less promising.

M. D. Van Wagoner seeks governorship of Michigan

Murray D. Van Wagoner, state highway commissioner of Michigan, has announced his candidacy for the Democratic nomination for governor of the state of Michigan. Van Wagoner, who received a civil engineering degree from the University of Michigan in 1921, was president last year of the American Road Builders Association. He was named state highway commissioner in 1933.

McLaughlin Air Service Photo

New Manhattan connection to George Washington bridge placed in service

Opened June 27, this concrete viaduct winds upward through Highbridge Park at the upper tip of Manhattan on a series of arches, recalling early Roman construction, to connect the Harlem River Speedway with surface streets in this area and a 2,400-ft. tunnel connecting directly with the George Washington Bridge. The tunnel, completed in 1931 but not previously used due to lack of outlets, crosses Manhattan under 178th St., and will carry two-way traffic on a 22-ft. roadway.

Only a few feet of the entire length of the viaduct is on tangent, the curves are of varying radii, maximum grade is about 8 per cent, and all curves are superelevated. Total cost of the recent work is about $2,000,000, of which 45 per cent was a PWA grant.

The viaduct was designed and its erection supervised by the Port of New York Authority under the direction of J. C. Evans, chief engineer, and Ralph Smillie, design engineer. Major construction was done by the North-Eastern Construction Co. of New York City.

A.S.T.M. meeting

(*Continued from p. 29*)

fication developments in manufactured masonry units occurred at the meeting. The study of brick disintegration under frost action continues at the Bureau of Standards. Three miles of paving, using different kinds of brick, are being studied in Ohio as background for specification revision. Studies are also under way looking to a specification for smooth facing tile for buildings.

Clay Pipe—Approval was given to incorporation of the higher minimum crushing strength requirement in the standard specification for sewer pipe. The specification covering definition of terms was withdrawn, as serving no useful purpose.

Fire Tests—New methods of fire tests of door assemblies were approved by the meeting. Revisions were made in the specifications for fire retardant properties of wood for scaffolding and shoring and in the fire tests of building construction and materials, to remove inconsistencies, to simplify the procedures, and to bring the methods up to date.

Steel—The always extensive specification activity of the steel committee was largely devoted to materials for high temperature service and to forgings. A new subcommittee is studying a program to aid in testing for weldability of steel.

Gypsum—Among numerous detail changes in gypsum specifications, perforated lath received new recognition. A new definition places the minimum diameter of holes at ¾ in. and states that they must comprise at least 2.75 per cent of the total area. Specification C 37-37 on gypsum lath was amended to include perforated lath.

Lime—No new lime specifications were presented. During the year an attempt to arrive at an agreement on specifications with the Plastering Contractors Association failed, due in part to a division of opinion in the lime committee over present A.S.T.M. specifications.

Mortars—Although no new specifications for mortars for unit masonry were presented it was announced that a program of research on shrinkage and expansion of mortar has been drawn up, and only awaits necessary funds. The committee is also studying specifications and methods of test for mortars for glass block which have been completed by a subcommittee.

Concrete pipe—Proposed revisions in the concrete pipe specifications admit smaller sizes (12, 15 and 18 in.) for concrete sewer pipe, while the maximum size of reinforced concrete pipe for sewers and reinforced concrete pipe for culverts is increased from 84 to 108 in.

Lucas & Kevorian Photo

Franco Photo

New honorary members

From top to bottom, Cloyd M. Chapman, H. E. Smith, and Thomas R. Lawson. Chapman is noted for his contributions to the study of concrete aggregates, cement, paints and oils. Smith, retired New York Central engineer, is a veteran of railroad testing and steel standardization. Lawson, of Rensselaer Polytechnic Institute, is noted for brick research.

Concrete—Two important methods of test in the field of concrete and concrete aggregates were announced: Test for efficiency of materials for curing concrete, and test for volume change of cement, mortar and concrete. Both of these methods of test are results of a definite demand that is an outgrowth of research which has been carried on for a number of years. The committee is giving much thought to future research work, some of the subjects being plaster of paris capping of test cylinders; moisture standards for the moist room; new methods of compression testing; method for determining water gain; method of sampling wet concrete; effect of speed of test loading on concrete properties; method of making concrete test specimens of concrete to be vibrated; method for determining volume of air in concrete; method of defining and determining soft particles in aggregate; and a specification for crushed fine aggregate.

Road Materials—Three new specifications already adopted were presented for the first time: (1) specification for preformed expansion joint fillers, (2) method of test for preformed expansion joint fillers, and (3) method of test for sieve analysis of mineral fillers. Two new specifications covering six grades of rapid-curing cutback asphalts and six grades of medium-curing cutback asphalts are ready to be presented to the committee on standards. Other specifications in this same final state cover slow-setting emulsified asphalt for fine aggregate mixes and traffic paint. Two new specifications adopted by the meeting cover materials for stabilized base course and for stabilized surface course of soil and soil aggregate mixtures; the types covered are sand-clay, graded coarse aggregate, gravel, stone or slag screenings, and sand. Because the Los Angeles abrasion test for gravel has practically replaced the old Deval test, the specification covering the latter was withdrawn.

Soils—Three new test methods covering soil-cement mixtures were adopted. These cover moisture density relations, a wetting and drying test, and a freezing and thawing test. A proposed method of test for water absorption and resistance to plastic flow of mixtures of soil and emulsified asphalt, approved by the committee, was rejected by the meeting and referred to the committee on standards as a result of a disagreement that developed among various commercial interests affected by the specification. The soils committee is at present studying the need for revising nearly all of its tests covering the physical characteristics of soils. It also has much research under way covering the consolidation test, shear tests, and stabilometer tests.

Fatigue of metals—This committee reported on an investigation that it has

under way to study the effects of the type of testing machine used and of the size of specimen. Aluminum, low-alloy steel and heat-treated steel have been investigated, using four types of machine —simple rotating beam, rotating cantilever, vibratory flexure and tension-compression. The principal conclusion to date is that aluminum is least sensitive to the machine used and the specimen size, that low-alloy steel is next, and that heat-treated steel is most affected.

Marburg lecture on cement

P. H. Bates of the Bureau of Standards, chosen Edgar Marburg lecturer for the present meeting, discussed portland cement theories and specifications. He pointed out that many essential facts about cement constitution and behavior are yet unknown, and that "there is not only an outstanding lack of the needed physical tests but also an equally outstanding lack of effort to develop such tests." Present methods of discriminating between cements are often ineffective, he said, as in the case of some cement that show good weather resistance in spite of high computed calcium aluminate content.

Of the new specification for five cements he said that it will enable the user to obtain more satisfactory service. Even though a type 5 cement used in sea water construction will not give positive assurance of long successful service, it is very likely to assure longer life than if cement of another type were used. He then discussed questions presented by the trend toward fine grinding, by adoption of the autoclave test and by problems of weathering resistance. Finally, he discussed admixtures, new tests, and variations in plant processes, and expressed the view that each of these fields offers opportunity for research.

Technical papers and discussions

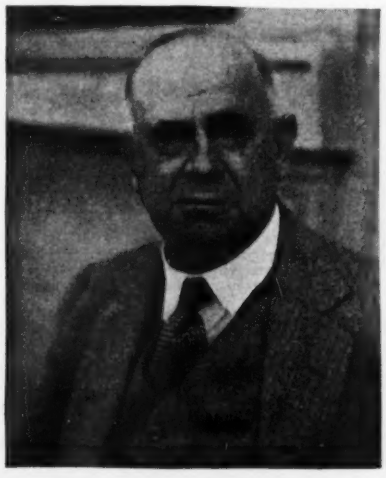
Supplementing the consideration of standards, about 100 technical papers were presented to the meeting, most of them reporting new data on properties of materials obtained through research. A number of newly developed test instruments and methods were described. Abstracts of the more important papers presented at the convention bearing on construction use of materials will be given in our July 18 issue.

Some of the committees also reported research results or service observations of considerable importance. Two exposure tests of iron and steel sheets initiated many years ago by the Corrosion Committee which came to an end during the past year verified earlier indications that copper-bearing metal resists rusting better than non-copper-bearing metal in atmospheric exposure, while in total immersion the copper-bearing metal has no clear advantage.

R. F. Walter dies; reclamation engineer

Raymond F. Walter, chief engineer of the U. S. Bureau of Reclamation since 1925, died in Fresno, Calif., on June 30.

Walter was born in Chicago in 1873 but his family moved to Fort Collins, Colo., when he was a boy, so that the major part of his life was spent in the semi-arid regions of the West. He graduated from Colorado Agricultural College as a civil engineer in 1893 and soon joined the staff of E. E. Baker, a civil engineer with an extensive practice in irrigation work. In 1899 he was made city engineer of Greeley, Colo., serving for two years, and then went into private practice as an irrigation engineer. From 1900 to 1903 he served as deputy state engineer in charge of water. In 1903, following the passage of the National Reclamation Act of 1902, Walter was engaged by the Reclamation Service to report on several irrigation projects in the Black Hills, and remained there five years. While there he designed and supervised construction of the Belle Fourche irrigation project. In 1908 he was made supervising engineer of the Rocky Mountain Division of the Reclamation Service, with headquarters at Denver, which position he held until the reorganization of the service in 1915, when he became senior engineer of the Bureau of Reclamation.



R. F. Walter

Such projects as Pathfinder Dike and the distribution system for the third unit of the North Platte project, the East and West canal systems and the Selig Canal system of the Uncompahgre project and the enlargement of Gunnison Tunnel were done under Walter's supervision during this period, as was construction of the Grand River Diversion Dam and the Canyon canal and tunnels of the Grand Valley project.

In 1917 Walter was made assistant chief of construction of the Bureau; in 1920 he became assistant chief engineer, and four years later he succeeded F. E. Weymouth as chief engineer. During his long service in this position he directed the planning and construction of all the great works for which recent Reclamation construction is notable, including Owyhee and Boulder Dams, All-American Canal, Grand Coulee and the Central Valley projects.

Planning Board Names three assistant directors

Frank W. Herring, for the past five years executive director of the American Public Works Association, Thomas C. Blaisdell, Jr., economist with federal government since 1933, and Ralph J. Watkins, former director of the Bureau of Business Research, have been appointed assistant directors of the National Resources Planning Board, to work with Charles W. Eliot 2nd, director of the board, in carrying out the functions of the Federal Employment Stabilization Act of 1931, recently assigned to the board by Congress.

Appointment of the assistant directors is part of a general reorientation of activities made by the board in view of the requirements of the act, which include preparing an annual six-year program of federal public works, reporting on economic and construction conditions in the United States, and advising the President of the trend of business and the approach of a depression.

For economic studies the board will develop a staff and will cooperate with other federal agencies that collect economic data. Information on advance construction plans of public and private agencies is to be gathered by the board's field offices and consultants. It is intended to continue development of plans and public works programs in cities, states and regions.

Public works programming is to be carried out in close collaboration with the Bureau of the Budget, and will give special attention to projects involving land, water and transportation. This division will be headed by Herring, recently executive director of the American Public Works Association and prior to that an associate editor of *Engineering News-Record*.

An executive order of the President issued on June 27 provides for close coordination between the 60 or 70 construction agencies of the federal government and the planning board and Bureau of the Budget. It requires reports by the agencies on project investigations, plans, estimates and actual construction. The order is expected to make the federal advance programs more definite than heretofore and increase their usefulness to development planning and to budgeting.

OBITUARY

W. F. HARRIS, 57, former city service director and engineer for the water department and board of education at Toledo, Ohio, died June 25, in Miami, Fla. Harris, a graduate of the University of Michigan, assisted in the construction of Toledo's present water filtration plant. He was associated at one time with A. Bentley & Sons, and for many years was a partner in the firm of Harris & Tansey, both general contracting organizations. He was service director at Toledo from May 1938 to September 1939.

LAURANCE JOHNSON CARMALT, 73, consulting engineer of New Haven, Conn., died June 24. Following his graduation from Yale in 1888, he worked with the Norfolk and Western R.R., the Southern Ry., New York Central, the Long Island R. R. and the New York, New Haven & Hartford R. R., being with the latter line from 1911 to 1921. Later Carmalt was made chairman of the original zoning commission in Hamden. He also worked with the U. S. Engineer Corps in Puerto Rico for a time.

CHARLES BENSCHOFF STEEL, 51, assistant professor of highway engineering at Pennsylvania State College, died at Williamsport, Pa., June 29.

JOSEPH A. TERTELING, 79, pioneer Idaho contractor, and member of J. A. Terteling & Sons, died June 28 at Boise, Idaho. Terteling's firm, organized in 1918, carried out many highway and irrigation projects in Washington, Oregon and Idaho.

GEORGE COOK, SR., 79, pioneer Minneapolis general contractor who organized in 1885 the firm of Pike & Cook Co. and remained head of that company until retiring in 1936, died recently in Pasadena, Calif.

NATHANIEL G. JOHNSON, 59, superintendent of the Burlington (N.J.) City Water Works, for the last 13 years, died in Burlington June 29.

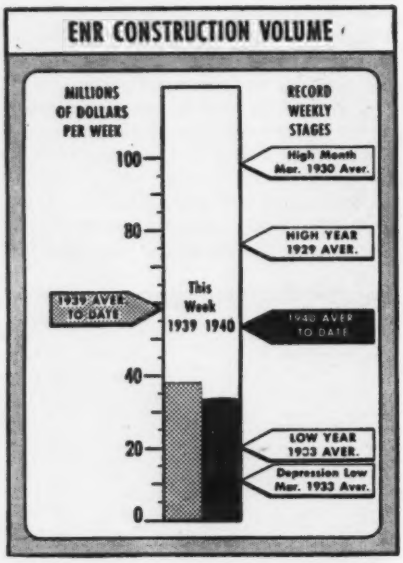
CLAUDIO CASTRO, 69, veteran Mexican civil engineer and dean of the National School of Engineering, died in Mexico City June 18.

HOWARD H. FETZER, 47, formerly engineer of Ashland County, Ohio, was killed June 21 near Sullivan, Ohio, while surveying a farmland plot that required setting up his instrument on the highway.

CONTRACTS AND CAPITAL

ENGINEERING CONSTRUCTION awards for the short week due to the Fourth of July holiday total $31,599,000, 16 per cent below the volume for the corresponding week last year.

Private construction for the week tops the 1939 week by 6 per cent, but public construction falls 21 per cent below a year ago.



The current week's awards bring 1940 construction to $1,421,215,000, a level 11 per cent below the 27-week period last year. Private engineering construction maintains its 24 per cent gain over a year ago, but public awards are 23 per cent below the total for the period last year.

In the classified construction groups, waterworks, bridges, streets and roads and unclassified construction top their respective totals for the preceding week; and bridges, industrial buildings, streets and roads and unclassified construction exceed their last year's figures.

New capital for construction purposes for the week totals $206,421,000, an increase of 40 per cent over the volume for the corresponding week last year. The addition of $191,000,000 in federal money for highway construction is responsible for the large volume and the gain over last year. Aside from the federal-aid money, there is $12,378,000 in state and municipal bonds, $3,007,000 in corporate securities, and $36,000 in RFC loans.

New construction financing for the year to date, $838,956,000, is 24 per cent below the $1,098,876,000 reported for the 27-week period a year ago.

(Weekly table on page 27)

Engineering Construction Contracts Reported by Engineering News-Record, in June, 1940

Engineering News-Record *reports projects of the following minimum costs:—water-works, excavation, drainage and irrigation,* $15,000; *other public works,* $25,000; *industrial buildings,* $40,000; *other buildings,* $150,000

Four Weeks — Thousands of Dollars (000 Omitted)

Public Works	New England	Middle Atlantic	South	Mid West	West of Mississippi	Far West	United States June 1940	United States Six Months 1940	United States Six Months 1939	Canada
Waterworks	124	798	1,177	1,739	725	223	4,786	39,236	112,263	371
Sewerage	469	1,741	188	1,220	1,250	1,155	6,023	44,951	96,058	.....
Bridges, public	1,431	1,332	796	846	1,449	778	6,632	50,681	90,604	237
Earthwork and waterways	455	1,973	1,138	4,368	1,260	3,616	12,810	124,821	101,500	.....
Streets and roads	2,334	20,163	9,575	12,793	20,441	5,481	70,787	364,041	339,534	2,508
Buildings, public	1,312	9,551	2,282	5,437	901	4,132	23,615	176,267	342,721	1,385
Unclassified, public	426	1,515	2,468	760	24,344	6,553	36,066	105,375	90,396	4,106
Total public	6,551	37,073	17,624	27,163	50,370	21,938	160,719	905,372	1,173,076	8,607
Federal government (included in above classifications)	623	3,512	3,891	4,691	25,236	11,881	49,834	215,431	150,419	.....
Private										
Bridges, private	.....	4,000	1,184	.....	.....	.....	5,184	8,471	2,362	.....
Buildings, industrial	8,965	7,753	17,291	6,638	5,332	5,151	51,130	222,453	103,936	10,093
Buildings, commercial	2,520	11,206	4,380	6,675	2,092	2,439	29,312	196,405	185,550	760
Unclassified, private	260	2,891	286	265	2,458	258	6,418	56,915	98,300	.....
Total private	11,745	25,850	23,141	13,578	9,882	7,848	92,044	484,244	390,148	10,853
Total Engineering Construction:										
June, 1940 (4 weeks)	18,296	62,923	40,765	40,741	60,252	29,786	252,763	.....	.....	19,460
May, 1940 (5 weeks)	13,401	64,557	75,792	54,315	49,714	24,517	282,296	.....	.....	20,180
June, 1939 (5 weeks)	11,200	85,358	48,996	55,819	38,317	28,705	268,395	.....	.....	12,880
Six Months — 1940	69,787	353,857	275,818	250,818	278,707	160,629	.....	1,389,616	.....	74,842
Six Months — 1939	86,181	513,860	241,209	296,609	278,100	147,265	.....	.....	1,563,224	44,312

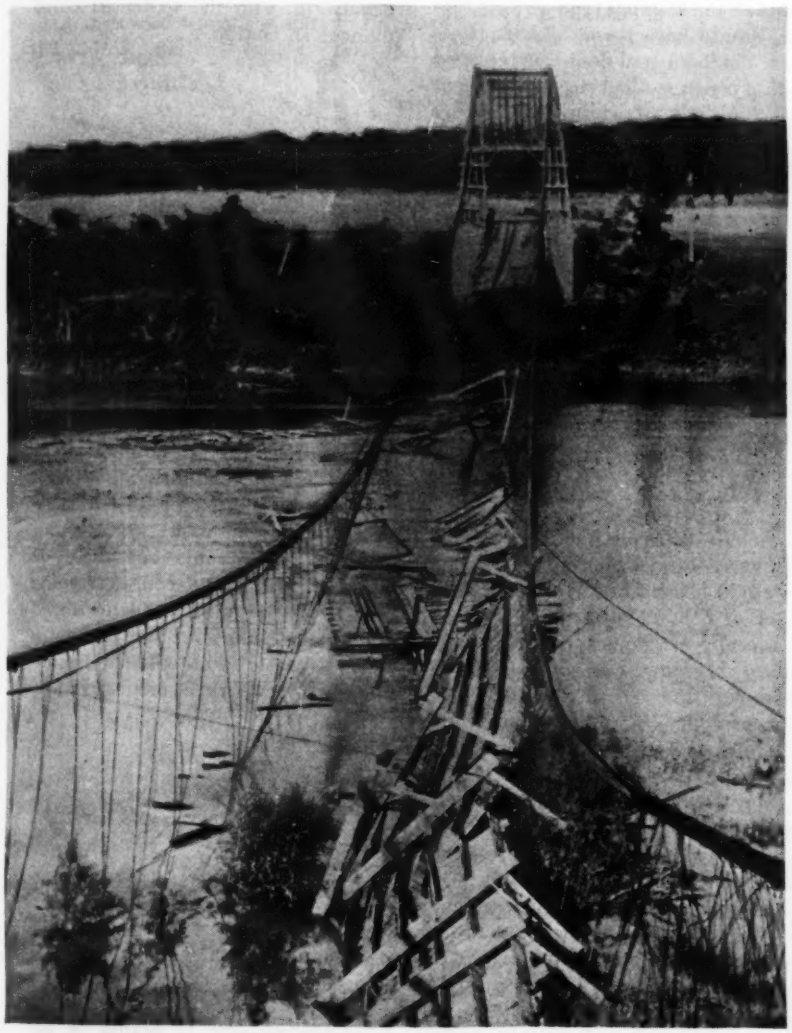
Manteno hospital head found guilty

In the typhoid fever case arising from the death of 60 inmates of a state institution at Manteno, Ill. (*ENR* Oct. 26, 1939, p. 517; April 18, 1940, p. 552), H. L. Bowen, state director of public welfare, has been found guilty of gross omission of duty by the Kankakee County Circuit Court. A $10,000 fine and removal from office is the maximum penalty.

The epidemic, the judge agreed with the prosecution, was caused by a contaminated water supply and Bowen ignored repeated warnings given by the state division of health. The judge indicated he did not believe the theory of the defense that the epidemic was caused by a typhoid carrier. July 11 is the date set to hear a motion for a new trial.

Chicago jury acts on ready mix concrete

A federal grand jury in Chicago June 24 indicted Mike Carrozzo, who is said to control 25 unions, and nine other labor union leaders on the charge of preventing the use of ready mixed concrete through strikes and threats of strike. Chicago is the only big city where ready mixed concrete cannot be used.



Cribbing failure drops bridge into river

The Brown Ford Bridge across the Osage River on a farm-to-market road in St. Clair County, Mo. six miles east of Lowry City collapsed June 21 while being remodeled. The repairs consisted of replacement with steel of wood parts at the towers. Collapse of timber cribbing under one of the suspension cables at the north tower (not shown in above view) is blamed for the accident. Three people were killed, including E. A. Bledsoe, 65, of Osceola, Mo., who constructed the bridge 25 years ago and was in charge of the repair work.

Defense training on S.P.E.E. program

Content of curriculum, methods of presentation, and relations of engineering training to industry studied

The annual convention of the Society for Promotion of Engineering Education, held at the University of California, Berkeley, Calif., last week, functioned effectively as a forum at which more than 500 engineering educators laid out plans for the coming year, discussed common problems and compared notes on improved methods and trends.

The program, despite its wide diversification to cover various branches of civil, electrical, mechanical and chemical engineering, with numerous attendant specialties, treated subject matter of three general kinds; namely, (1) content of the curriculum, (2) methods of presentation and (3) relations of engineering training to industry.

Of outstanding interest in the special sessions was a paper by W. F. Durand, Stanford University, reviewing engineering requirements in the aviation industry. Describing first the several types of modern aircraft used in warfare and the difference in their functions, he pointed out what the manufacturing requirements were and stressed the importance of training personnel for instruction in three related branches of engineering education; namely, (1) aerodynamics (a special branch of fluid mechanics), (2) structural design and (3) mechanical engineering.

O. J. Ferguson, president of the University of Nebraska, reported progress in training personnel for national defense. On request from Owen D. Young of the National Defense Council, joint meetings have been held by representatives of various federal departments and ten educational organizations, including S.P.E.E., at which plans for this special training have been discussed.

The regular instruction in engineering colleges is not to be interrupted but will be supplemented by emergency training designed to meet defense preparedness requirements. In agreement with federal officials, the special training program has been planned (*ENR* June 20, 1940, p. 839) for the present in two levels, (1) sub-collegiate and (2) post-collegiate. Secondary schools are to handle the first of these and colleges the second.

Congress has appropriated $15,000,000, Ferguson reported, for the sub-collegiate training work, and an early appropriation to start the more advanced work is expected.

Officers elected for the coming years are: president, D. B. Prentice, Rose Polytechnic Institute; vice presidents, L. E. Conrad, Kansas State College and E. L. Moreland, Massachusetts Institute of Technology.

COMMENT AND DISCUSSION

Readers' opinions on matters that concern engineers and contractors

Geophysical Methods

Sir: The article by A. E. Wood in your issue of March 28, p. 438, "Damsite Surveying by Seismograph," is a useful reminder of the steadily growing appreciation of geophysical methods as an important aid in the sub-surface exploration work which is so essential a part of civil engineering practice. That the utility of geophysical methods in civil engineering is not restricted to preliminary investigations has been demonstrated recently in Ontario. A large crane was accidentally buried by an extensive clay slide and was finally located (under 15 ft. of clay) by means of a magnetometer survey, after exploration by test boring had proved fruitless. The crane was subsequently uncovered, removed and reconditioned. The same magnetic geophysical method was used for the survey of the newly discovered iron ore deposit, recently completed off the ice of Steep Rock Lake in northwestern Ontario.

R. F. LEGGET
Asst. Prof. of Civil Engineering
Toronto, Ont.

Bridge Fires

Sir: An editorial in *Engineering News-Record*, April 25, 1940, p. 570, implies that fires have not occurred in modern laminated timber decks. Perhaps your attention has not been called to the fire which occurred in September, 1939, on the laminated floor of the cantilever bridge over the Ohio River between Williamstown, W. Va., and Marietta, Ohio. This bridge, 2,450 ft. long, is made up of a cantilever main span with viaduct and truss-span approaches. The trusses are 28 ft. apart center to center, the space between consisting of a 21½-ft. roadway and a 4½-ft. sidewalk, there being an electric-railway track on one side of the roadway. The roadway floor consists of 3 x 4-in. planking laid on edge and supported on longitudinal wood stringers except under the car track where steel stringers are used.

The fire occurred on a Sunday afternoon from an unknown origin and spread over a length of 800 ft. near the center of the bridge before it was finally extinguished. The fire destroyed most of the laminated flooring, as well as some of the timber stringers. Many of the steel stringers were badly burned and had to be replaced.

The cost of the repair work on the basis of the original type of construction would have been $38,500. However, the laminated floor was replaced with a concrete-filled steel grid except for the 10 ft. of width occupied by the street-car track. The timber stringers were replaced with steel stringers. The total cost of repairs was approximately $52,000.

R. P. DAVIS, DEAN
College of Engineering
West Virginia University
Morgantown, W. Va.

Thin-Shell Dome

(Editor's Note. This letter was received from the late Zara Witkin shortly before his death on June 6.)

Sir: I was deeply interested to note the excellent description of a dome of a new church in Montreal in your issue of Feb. 1, 1940, p. 72. In fact, it is really two domes, one within the other. The outer one is about 118 ft. diameter and 96 ft. height, 7-in. thickness. This would make the ratio of thickness to diameter approximately 1 to 200.

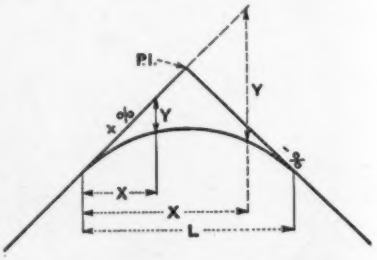
May I call your attention to your issue of Aug. 15, 1929, p. 257, in which the dome of the Wilshire Temple, in Los Angeles, is described. This dome was 104 ft. in diameter and 57 ft. high, with a thickness at the base of 4½ in. and at the crown of 2½ in., averaging 3½ in., a ratio to diameter of less than *1 to 300*. This dome was built of pneumatically placed concrete, using one compressor to the five used in Montreal. It supports a heavy inner false dome of coffered staffwork hung from the structural dome by steel braced hangers, thus forming a plenum chamber. The dome was shot with one gun with a special core-breaking nozzle, allowing re-mixing of the aggregates at the point of shooting. The hose was taken up outside with no necessity for cutting holes in the forms.

It must be remembered that the flatter a dome is, the greater the stresses and the difficulty of construction. The Montreal dome has a high ratio of height to diameter. The Wilshire Temple dome is relatively flat. The Wilshire Temple dome endured the severe earthquake of 1933 with no apparent damage. Occasionally, since, we have had some very noticeable tremors. Design of this dome was by Paul Jeffers, structural engineer, construction by H. M. Baruch Corp., with the writer as chief engineer in charge.

ZARA WITKIN
Los Angeles, Calif.

Vertical Curves

Sir: Professor Furr, in your issue of March 28, 1940, gives a method for figuring the elevations on a vertical-curve parabola. It seems to me that the method is subject to the handicap of carrying an initial or intermediate error in arithmetic through to the end of the computations. The real problem in engineering computations is to find, in a given instance, the shortest possible solution within the arithmetical bounds determined by the basic measurements. For the vast majority of vertical-curve computations, the slide rule gives sufficiently accurate results, and I am submitting herewith a method for computing the ordinates to a parabola with the minimum amount of slide rule computation.



Referring to the figure, let

A=Algebraic sum of grades in per cent

L=Length of curve in 100 ft. stations.

Y=Drop from tangent in feet

X=Distance in feet from point of tangency

Then, $Y = \frac{AX^2}{20{,}000L}$

To obtain the value of Y at a number of given points by one setting of the slide rule, set the value of $\frac{A}{20{,}000L}$ on B scale under the index on A scale, then set the indicator on the value of X on D scale and read the value of Y on B scale above. All possible values of X may thus be disposed of with two settings of the slide rule.

Or, the slide may be moved and the indicator left unchanged, reading thereby all values of X with one setting of the indicator, as follows: Set the indicator on the value of $\frac{A}{20{,}000L}$ on A scale and adjust the slide so that the indicator is also on the value of X on the CI scale. Then read the value of Y on the A scale above the index on B scale.

It should be noted that Y may be so calculated for any tangent to the curve, and the simplest way to figure elevations on a symmetrical curve is to calculate Y from a horizontal tangent at the summit of the curve. The distance in feet from the PI to the summit (the "middle ordinate" of the curve) is $\frac{AL}{8}$. From the elevation of the summit the elevations on the curve may be determined by subtracting the calculated values of Y, X being measured either way from the summit.

This method of figuring ordinates to a vertical curve requires but a moment's study to master, saves time over any method that has ever come to my attention, and reduces to a minimum the chances for error.

W. T. St. Clair
Nashville, Tenn.

Same Old Story

Sir: The discussion "What's an Engineer?" by W. K. Palmer on page 38 of the March 28 issue sounds very similar to others I have read during the past 20 years, and although the tune stays about the same the recording is played more often. It's a nice tune and I rather enjoy it, but wish we could get it out of the same class as Mark Twain and the weather. We should progress from the "just talking" stage, and with that in mind I shall offer one definite suggestion as to how we might "do something about it."

In my particular vicinity I know personally most of the professional engineers just as others do in their localities. When I see any mention, advertising, news item, or otherwise, of an engineer or engineering service with which I am not familiar, I take a few minutes to check the registration list, and if the alleged engineer is missing, I call the matter to the attention of the State Board of Engineering Examiners with a suggestion the situation be investigated. It really has brought results.

If this were done all over the country, we should at least have a start toward a housecleaning, and with not too much extra work for anyone. Incidentally, I believe the mechanicals have the biggest single problem in the air-conditioning field. It is swarming with "engineers."

In concluding, Mr. Palmer says, "Let the older engineer vociferously protest these evils!" How about letting the younger engineer in on the protesting? By all means let the older men continue to protest, but the younger men had better be getting in practice. They have more years to keep at it, and I'm afraid they'll need them.

But let's not wait for a general cure-all. Let's start the yeast on some of the little things, and maybe it will work up through the loaf.

L. W. Mahone
Assistant Professor, Iowa State College
Ames, Iowa

P. S. Some of your advertising copy could, at times, be improved along this line.

Building for Air Raids

Sir: It may be of interest to note that a new factor has entered into the design of buildings located in many parts of the world. Happily, this factor need not be considered for buildings located in the United States.

At a meeting of The Institution of Structural Engineers, December 14, 1939, at their headquarters in London, the subject for discussion was, "The Application of Air Raid Precautions to Future Buildings." The president's address and the discussion thereon is ably reported in the February 1940 issue of the *Structural Engineer*, the Journal of the Institution.

The president stated that the taking of precautions against air raids should be regarded as one of the essentials of building design, "as essential as the siting, the sanitation or ventilation of the building." To do this adds another factor to the problem of building design. The phases of this factor are many. One speaker emphasized that the purpose of camouflage was to mislead a raiding airman; "sometimes camouflage which appeared to be excellent when viewed from the ground could be very ineffective when viewed from the air."

In this connection the following from an article by the Military Correspondent of the *London Times* (*New York Sun*, November 11, 1939) will be quoted: "Watching an air raid, even if it seems to be a long way off, is not a safe occupation. Bullets and pieces of shell may fall two miles away from the actual scene of fight."

Robins Fleming
Structural Engineer (Retired)
New York, N. Y.

Saving Fish from Metal Poisons

Sir: A simple method which the writer found effective in protecting fish in exhibition tanks from metallic poisoning may be of use to some of your readers in the water purification field.

At the Steinhart Aquarium in San Francisco, brook trout died so quickly that it was impractical to keep them on exhibit. Fish that were perfectly healthy when put into the tanks soon went into spasms and died. Analysis showed the water to contain 1 ppm of zinc, picked up from the galvanized cooling coils. Fish are particularly susceptible to zinc poisoning because they have only a small amount of blood and in the gills the soluble poison has direct access to it.

Lime would be the logical material with which to neutralize the zinc but of course lime in the water would be injurious to fish. The problem was solved effectively by placing coarse-ground oyster shells in the water coming from the cooling coils. The shells collect, as a precipitate, solutions of metallic salts. In other words, the lime of the oyster shells is effective without putting any lime solution into the water of the tank. The shells require occasional replacement, of course, which is a negligible item as compared to complete solution of a troublesome problem.

John Finn
Chemist
San Francisco, Calif.

ENGINEERING NEWS-RECORD

Copyright 1940, by McGraw-Hill Publishing Company, Inc.

Volume 125, Number 1

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July 4, 1940

SINCE SPRING, contractors have been pushing paving work on the Pennsylvania Turnpike to get that 160-mile superhighway as near to completion as possible by the PWA deadline of July 1. Paving of a few miles got under way last fall, but when winter shut down there were 140 miles of four-lane road yet to be paved. That called for high speed this year. To complicate matters, all sand and cement had to be brought in for some distance and points of access to the right-of-way were limited. That made extensive advance stocking of materials necessary at such points along the road as could be reached. It called also for careful planning to integrate the movement of supplies and the operations of many contractors. The results stand as a credit to the engineers and contractors concerned. In these days when speed in constructing needed facilities is assuming unusual significance, the record made on the Turnpike renews confidence in the ability of engineers and contractors to organize to meet any emergency.

Inadequate Airports

PEOPLE who have had any contact with municipal airports over the country have known that most of them are inadequate for safe use by modern aircraft, but it was not until a great expansion of the nation's airforce was ordered that the seriousness of the airport situation was realized. An airforce that is to defend our shores effectively must have many fields from which it can operate safely; it must have access to commercial as well as military fields. That adequate commercial fields are lacking is not entirely the fault of our city governments; the present deficiency is due chiefly to the rapid advance in airplane development of the past few years that has made airports obsolete soon after they were finished. Some authorities now believe that most of the period of rapid change in airfield requirements is behind us, that economic factors will check the introduction of larger planes. Such being the case, cities now can plan with more confidence for modernization or replacement of their airports. To that end, we give on another page a summary of the requirements set up tentatively by the Civil Aeronautics Authority for airports for various types of commercial needs. What requirements the Air Corps will set up for the commercial fields that may be included in its planning is not yet known, but it seems quite probable that they will not differ sufficiently from commercial needs to keep municipalities from going ahead with much-needed airport planning.

Progress in Cement

ADOPTION of a specification for five types of cement by the American Society for Testing Materials last week marks an epoch in cement and concrete development. In contrast to the disagreement that brought a similar attempt to failure some years ago, the present step has the general approval of manufacturers and users alike. It represents final abandonment of the single-standard-cement doctrine that survived substantially unimpaired for 30 years. But more than this, it reflects a spirit of progress from which accelerated improvement of concrete is sure to follow. Henceforth the engineer and builder in concrete will have available a moderate-heat and a high-early-strength cement, and on special order also a low-heat cement and a sulfate-resisting cement, in addition to the normal or general-purpose cement. If proper use is made of this enlarged range of materials, concrete can be adapted more closely to service requirements than has been possible in the past, and better structures will be built, especially if concrete men by reporting their experiences with the several cements provide adequate data to confirm the new specifications or else guide in their revision.

Further Possibilities

BY AN INTERESTING COINCIDENCE, P. H. Bates in the Edgar Marburg lecture pointed to various possibilities for further improvement of cement. His frank words on the large number of unknowns in the constitution and behavior of the complex substance that we know as portland cement, and his admonition that we explore all possible discriminatory tests and all means for modifying the material (admixtures, for example) indicated clearly that there is room for both fundamental scientific study of cement and development of

better ways of using it. The tribute paid by the lecturer to the excellence of present-day cement added emphasis to his statements. Nothing that he said gave reason to anticipate major change in the five-cement classification now established, at least for some years to come. He did indicate, however, that details of production and use of cement can still be refined, and that such refinement promises to give us better cement, better concrete and more certainty in attaining the desired service results.

Those Bottlenecks

THERE IS MUCH TALK of bottlenecks in our defense preparations which, if examined in the light of reality, is seen to be nonsense. In other words, the bottlenecks are theoretical and exist only in the popular mind and not at all in the minds of our defense leaders, civil or military, who know the realities of the situation.

To assume that we are going to effect a production rate of 50,000 planes a year in the next few months is not reality; it just can't be done. To assume that we can be turning out huge quantities of powder, shells and bombs in a year is equally unreal; there are too many things to be done to get ready for such munitions manufacture. Despite all that we could do in 1917 the war was nearly over before we could supply our own army with American munitions. There is no reason to suppose that we can greatly improve on our earlier record.

Therefore, by the time we need more manganese we will be able to get it from Cuba, Brazil or Nevada, should other sources be closed. Before we have to face a tin shortage we will be eating out of some other kind of cans than tin, and thus can conserve this valuable material. By the time we can use more armor plate, aluminum and chemicals, the plants will be ready to turn them out. By the time the bombs and personnel are available the theoretical bottlenecks in airplane engines and machine tools will be licked. By the time we run out of natural rubber, new synthetic rubber plants can be ready to take over.

The reality of the situation is that we must count on a two-year program to put our national defense in order. On such a schedule, by the time we reach the bottlenecks they will have disappeared.

The significance of all this to the construction industry is that efficiency in the building of munitions plants, engine and plane factories, airbases and shipyards, will make production bottlenecks impossible. Quick organization by the construction industry for the task ahead will eliminate the development of any bottlenecks in its own field of operations.

Are Military Roads Needed?

THE REASONING that causes Congress to cut federal aid appropriations for highways at this time is completely obscure. If roads are essential to the national defense it is hardly logical to curtail their construction while lavishing money on every other type of defense expenditures.

So far as is known the War Department still subscribes to its theory that roads developed for commercial purposes are adequate for military needs. But such a statement is ambiguous. It does not make clear how well developed the roads should be. In view of the recognized inadequacy of our roads for large volume commercial traffic, it certainly does not mean that present roads are adequate.

Presumably, what it means is that the 27,000-mile road network that the PRA proposes as our basic system is adequate; but that will take 25 years to develop under the present federal-aid act. Is that speed of progress satisfactory to the War Department? If so, everything is fine and we can turn our attention and money to more vital defense measures. If it is not, the War Department should say so, and give Congress some hint of what is needed.

It is true that the Army has recently expressed some concern over inadequate bridges and, as a result, pressure is being placed on state highway departments to give priority to bridge work. It is true also that WPA money is being earmarked for access roads to army posts and airbases. But what about the projected roads on the War Department map that supplement our existing coast and border highways? Not many months ago, these roads would have been discussed only in whispers. But times have changed. Is the Army satisfied for these to have leisurely development in accord with commercial needs alone?

If we need more roads and better roads for military purposes, why aren't we frankly planning them and setting aside the money to build them? A frank statement from the General Staff would aid Congress and would end much uncertainty on the part of state highway departments as to what plans they should be making in view of the present emergency.

Basic Requirements for Airport Design

FIG. I. THE MUNICIPAL AIRPORT AT TAMPA IS LOCATED IN THE BAY CLOSE TO THE CENTER OF THE CITY.

Contents in Brief—More and better airports are among the most urgent needs of the country at the present time due to the rapid obsolescence of existing air fields and to a great expansion of military requirements. This article sets forth some of the fundamentals that must be taken into consideration in planning new airports or in modernizing existing ones.

RAPID INCREASES in the size and speed of airplanes have made most of the nation's airports obsolete for commercial purposes. Now the need for a vast military air force that can be concentrated at almost any point along our thousands of miles of coast lines and borders makes our lack of adequate airports a matter of major national concern.

Last year the Civil Aeronautics Authority made a comprehensive study of the airports of this country which showed that no single airport came up to what it believed to be the minimum requirements for a major air terminal and only 36 came up to the standards set tentatively for cities of over 25,000 population along main airways. The CAA found also that 3,500 airports were needed to serve the country adequately but that there were less than 2,200 airports and intermediate air fields. This survey was made before conditions in Europe made rapid expansion of our military air forces necessary.

The Army and Navy already have in hand the work of building great new or expanded air bases that will serve as the home ports for their air forces, but these forces must be able to use many of our municipal airports if they are to function effectively. This puts squarely up to municipal authorities the need for prompt action in bringing their airports up to standards that will serve both military and modern commercial requirements.

What then are some of the basic requirements for these modernized airports?

At the outset it should be said that no plans for the modernization of a municipal airport should be carried far without establishing close contact with the regional offices of the Civil Aeronautics Authority and the military authorities. Also, it is desirable to establish contact with the state or regional planning authority so that there will be no wasted effort through building airports of a higher classification than is needed for a unified airport system for the region. And of course, in the case of cities already served by one or more of the commercial air transport companies, officers of those companies should be approached for information as to their future plans and probable needs.

Classes of airports

For convenience in estimating the requirements for various types of communities, the airport section of the Civil Aeronautics Authority has set up tentatively four classes of airports for planning purposes. They are given in Table I. Briefly, they range from Class 1, the small airport for communities under 5,000 population, to Class 4, the large airports for major terminals at the large centers of population. The classifications have been made by the Civil Aeronautics Authority for planning purposes and have only been adopted tentatively pending a final determination of airport rating standards.

A common mistake of the past has been the selection of a site for an airport that was so limited by paved streets, highways, railroads or other permanent developments as to make future expansion uneconomical or impossible. The result of this is that many airports cannot be expanded to

meet present requirements but must be replaced by new air fields in locations where they can be used safely by present-day planes and where expansion to meet future needs will be possible within reasonable cost.

The ideal airport for the small Class 1 community, as laid out by the Civil Aeronautics Authority, would be a quarter-mile square so situated as to permit its expansion to a full square mile by the addition of adjoining quarter-mile squares as future need develops. At the time of the initial purchase, options should be taken on the whole square-mile area and the adjoining region should be zoned to prevent the erection of structures that would limit access to the air field. Model zoning ordinances for this purpose can be obtained from the CAA.

The minimum field for a Class 1 airport is shown in Fig. 2 and its ultimate expansion to meet the Class 4 requirement is shown in Fig. 3. Obviously, few communities except

TABLE I—AIRPORT SIZE STANDARDS FOR USE IN PLANNING

	Class 1 Airport	Class 2 Airport	Class 3 Airport	Class 4 Airport
Type of community	Small communities not on present or proposed air carrier system. Includes communities up to a population of approximately 5,000.	Larger communities located on present or proposed feeder line airways and having considerable aeronautical activity. General population range 5,000 to 25,000.	Important cities on feeder line airway systems and many intermediate points on the main line airways. General population range 25,000 to several hundred thousand.	Cities in this group represent the major industrial centers of the nation and important junction points or terminals on the airways system.
Type of plane which airport may safely accommodate.	Small private owner type planes up to gross weight of 4,000 lb., or those whose wing loading times power loading does not exceed 190.	Large private planes and some small transport planes in gross weight range between 4,000 and 15,000 lb., or having a wing loading × power loading of 190 to 230.	Present-day transport planes between 10,000 and 50,000 lb. gross weight, or having a wing loading × power loading of 230 and over.	Largest planes in use and those planned for immediate future with a gross weight of 50,000 lb. and over, or a wing loading × power loading of 230 and over.
Length of landing strips*	1,800 to 2,500 ft	2,500 to 3,500 ft	3,500 to 4,500 ft	4,500 ft. and over.
Width of usable landing strips	300 ft.	500 ft.	500 ft.	500 ft.
Length of paved runways	None	2,500 to 3,500 ft	3,500 to 4,500 ft	4,500 ft. and over.
Width of paved runways	None	150 ft., night operation. 100 ft., day operation only.	200 ft., for instrument landing. 150 ft., night operation. 100 ft., day operation only.	Same as Class 3.
Number of landing strips and runways determined by percentage of winds, including calms, covered by landing strip and runway alignment.**	75 per cent	80 per cent	90 per cent	90 per cent.
Distance between center lines of parallel runways.			700 ft. minimum	700 ft. minimum.
Distance between center line of runway and airport buildings or aprons and loading platforms (instrument landing runway).		700 ft. minimum	700 ft. minimum	700 ft. minimum.
Distance between center line of runway and airport buildings or aprons and loading platforms and parking areas (other runways).		250 ft. minimum	250 ft. minimum	250 ft. minimum.
Landing strip and runway grades — transverse.	2¼% maximum	2% maximum	1¼% maximum	1½% maximum.
Landing strip and runway grades — uniform longitudinal.	2% maximum	2% maximum	1½% maximum	1½% maximum.
Grade breaks — longitudinal***	Minus 1½% to plus 1½% maximum.	Minus 1½% to plus 1½% maximum.	Minus 1% to plus 1% maximum.	Minus 1% to plus 1% maximum.
Static design loads for runway and apron paving based on present day aircraft. Load considered distributed equally between two main wheels or sets of wheels.	No paving recommended	30,000 lb	60,000 lb	100,000 lb.
Probable future (10 years) maximum static gross loads to be considered in the design of runway and apron paving of drainage structures.	No paving recommended	60,000 lb	150,000 lb	300,000 lb.
Recommended design impact factor for paving to be applied to gross weight.****	No paving recommended	2	2	2.
Recommended design impact factor to be applied to paving loads per unit area.****	No paving recommended	1½	1½	1½.
Probable range of static airplane tire pressures.	10 to 25 lb. sq.in	15 to 50 lb. sq.in	30 to 75 lb. sq.in	50 to 85 lb. sq.in.
Facilities required	Drainage, fencing, marking, wind direction indicator.	Marking, wind direction indicator, drainage, lighting, hangar and shop, fueling, fencing, weather information, office space.	Marking, wind direction indicator, drainage, lighting, hangar and shop, fueling, fencing, Weather Bureau, two-way radio, visual traffic control, instrument approach system — when required, administration building.	Same as Class 3.

* All of the above landing strip and runway lengths are based on sea-level conditions; for higher altitudes increases are necessary. One surfaced runway is recommended for the full length of each landing strip for airports in Classes 2, 3, and 4.

** Landing strips and runways should be sufficient in number to permit take-offs and landings to be made within 22½ deg. of the true wind direction for the percentage shown above of winds over 5 mph, based on at least a 10-year Weather Bureau wind record where possible.

*** Longitudinal intersecting grades on a runway or landing strip should be joined by a vertical curve at least 500 ft. in length. It is also recommended that the distance between successive grade breaks be not less than 1,000 ft. In general, there should be no change in landing area grades of more than ½ per cent in any 100-ft. interval.

**** Allowance for impact in paving design may be made either in terms of gross load or applied load per unit area. Both allowances in combination, however, should not be used.

This table is based on a group of tables compiled by the Airport Section, Technical Development Division, Civil Aeronautics Authority, Washington, D. C., and published in "Airport Design Information."

those on the prairies will be able to attain this ideal, and even on the prairie prevailing winds or other physical conditions will call for major departures from the ideal. The site selected for most municipalities will have to be a compromise between the ideal and the local physical or economic limits.

For meeting these limits, the Civil Aeronautics Authority has set forth the fundamentals that should be taken into consideration. First, as previously noted, is the matter of ample space for future expansion. Closely related is the matter of freedom from large obstructions such as mountains, high hills or tall buildings within a 2-mile zone around the site and freedom from lesser obstructions such as trees, transmission towers and stacks within the area closer to the field. Accessibility and the relation of the proposed field to existing means of surface transportation also are major factors to be considered. The smaller communities, in selecting a location for an airport, should give thought to its relation to the routes of existing airways; larger communities, in selecting a new site, should take into consideration its relation to neighboring fields in the same region. The CAA states that for safe handling of air traffic, there should not be less than 6 miles between the centers of the landing areas of major fields.

Obstructions around airports

Obstructions such as trees, power lines, buildings and hills close to air fields reduce the effective length of landing area. This reduction is computed on the basis of approaches being clear of vertical obstructions within a glide path of 1 on 20 for the small Class 1 airports and 1 on 30 for the larger airports. For runways to be used for instrument landing, a ratio of 1 on 40 is required, this ratio to start at a point 4,500 ft. from the beginning of the runway. Thus an obstruction 50 ft. high in the "approach zone" to a runway close to the boundary of a large airport would call for an increase of 1,500 ft. in the runway length. The approach zone is defined as a trapezoidal area having a width of 1,000 ft. at the boundary of the airport and broadening to 4,000 ft. at a distance of 2 miles out.

Effect of altitude

Meteorological conditions have an important bearing on the selection of sites for air fields. In the higher altitudes, the lower barometric pressures call for material increase in the length of runway and in the zone that must be free from obstructions. These increases are given in Table II.

TABLE II — EFFECT OF ALTITUDE ABOVE SEA LEVEL ON AIRPORT SIZE

	Runway lengths in feet			
Elevation	Class 1	Class 2	Class 3	Class 4
Sea Level	1,800	2,500	3,500	4,500
2,000	2,040	2,840	3,980	4,890
4,000	2,340	3,250	4,550	5,250
6,000	2,690	3,740	5,230	5,670
8,000	3,120	4,350	6,070	6,150
10,000	3,660	5,090	7,120	7,120

Note.— The effect of altitude is less marked in the case of Class 4 airports since it is assumed that airplanes which will be critical in determining airport size in this classification will be equipped with supercharged engines.

Proposed sites must be carefully investigated as to wind, fog and smoke conditions that will affect visibility in the vicinity of the airport. Careful study should be made of the wind direction and duration, as the direction of prevailing winds materially affects the layout of runways and the placing of buildings.

Temperature is another factor controlling the length of runways. In regions where temperatures may be high, the decrease in air density resulting therefrom makes a longer run necessary for taking off and decreases the rate of climb.

Airplane performance

As the performance of aircraft is a very essential factor in determining the size of an airport, the aviation interests have established the practice of rating airplanes according to their wing loading and power loading. The wing loading is obtained by dividing the permissible gross load in pounds by the total wing area in square feet, and the power loading is obtained by dividing the gross weight of the plane by the total horsepower available. The stalling speed of an airplane increases as the wing loading increases, consequently the speed which must be attained prior to takeoff is greater and the minimum safe speed which must be maintained while gliding in for a landing also is greater. In general, as the power loading increases, the rate of climb decreases and hence a greater takeoff distance is required. The present trend in aircraft design is away from small airplanes with light wing loading to fast transport planes with heavy wing loading, higher landing speeds and slower climb ratios. Among the large planes now in use are the Boeing 307, gross weight 45,000 lb., and the Douglass DC-4, gross weight 50,000 lb.

Topography and soil conditions

Topography and soil conditions are major factors to be considered in the selection of a site and grading and drainage are large items in airport cost. The best sites are those that depart sufficiently from the dead level to provide adequate drainage without being too steep to result in erosion and those sites where runways of adequate length and width can be obtained without expensive grading. Sites of natural bowl formation, or those which are located on knolls where the land slopes gently in several directions, offer special advantages both for runway construction and for drainage. The preferable airport is one wherein all the area within the boundaries is safe and usable for aircraft, but such areas are not available in many parts of the country. Natural drainage that will cut the amount of artificial drainage to a minimum is especially desirable.

The type of soil at possible airports calls for careful study. The most desirable types are those containing a reasonable amount of porous materials such as gravel, sand or decomposed granite combined with a suitable natural binder. Undesirable types are those that become plastic and have a low bearing power when wet. It is essential that airports be usable throughout the wet seasons of the year, hence a soil investigation should include a determination of the maximum height of groundwater level; sites having a high groundwater level are costly to drain. Sites lying in river valleys should be carefully studied as to possible flood heights.

Grading materials

Other factors to be considered are the availability of suitable materials for grading purposes if the materials of the site are unsuitable, and the nearness of power and water supplies and sanitary sewers. In northern regions where snow is to be expected through a considerable part of the year, the drifting of snow and the large runoff resulting from melting snow in the spring must be considered in the layout of an air field.

The soils of an airport should be capable of carrying the applied loads under all weather conditions, hence a careful study of the soil characteris-

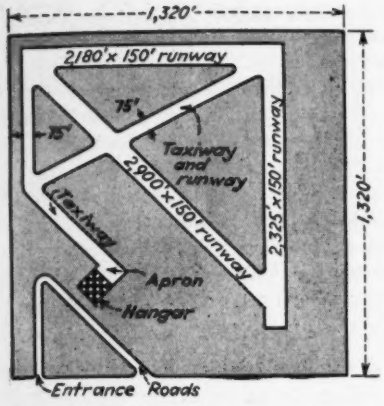

Fig. 2. First unit of the stage development of a quarter section for a small Class 1 airport.

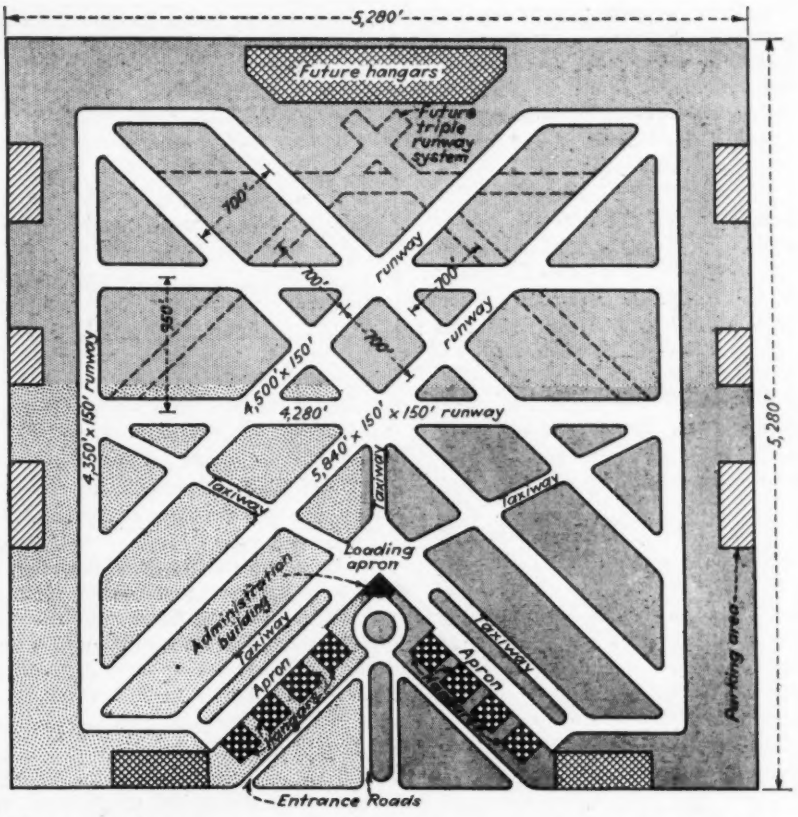

Fig. 3. Master plan for a major airport showing the three stages of development. The dark shaded area at the lower right shows how the initial development of a quarter-mile square is fitted into the larger plan.

tics is essential. If the natural soil has a low bearing value soil stabilization may have to be resorted to in order to reduce the cost of runway surfacing.

Drainage

Because drainage may become a major factor in the cost of airport construction, it calls for careful study in the comparison of available sites. As a basis for the design of drainage structures, available rainfall and runoff records for the region must be studied. For most sections of the country, use of the 10-year 1-hour maximum rainfall curve represents conservative drainage design practice and is recommended by the CAA. In regions of heavy snowfall, the design studies must include the combined effect of heavy rainfall and melting snow on frozen ground.

The most important factors to be considered in the design of a surface drainage system are the relative permeability of the various surfaces to be drained, the gradient over which the surface water must flow, and the rate of flow over these surfaces. The percolation through and permeability of paved runway surfaces, taxi-strips and aprons, can be considered negligible. For general practice, runoff from such surfaces can be considered as 85 to 100 per cent of the total rainfall. In determining the runoff from unpaved areas, a positive measure of the permeability of the soil offers the only rational method for determining the quantity of water to be handled. The ideal site is one in which the major part of the drainage can be handled over the surface without erosion, but some artificial drainage must be provided at any site, particularly for the larger air fields. As the drainage costs on some air fields have been as high as $300 to $400 per acre, the matter of drainage may be a controlling factor in the selection of a site. Where the soil on the site is found to be of impervious material, extensive sub-drainage by artificial means may be found necessary.

Grading and runways

After a site has been selected, a rough grading plan of the entire area to be included in the ultimate development of the airport should be prepared as a preliminary to the detailed grading plan for the section that is to be developed initially. Such a preliminary plan for the entire area is essential if costly grading and major changes in the drainage system are to be avoided when the field is enlarged.

Grades on longitudinal and transverse landing areas and runways of major airports should not exceed 1½ per cent unless that objective cannot be attained at a reasonable cost. On the other hand, perfectly level grades should be avoided except on very pervious sand and gravel in order to simplify the drainage problem. A grade of about 1 per cent will provide for a quick runoff without being objectionable from the operating viewpoint. In general, surface drainage water should be carried away from paved areas and building sites.

Airport surfaces

Firm turf surfaces may be used for airports with light and limited traffic in regions where soil and climatic conditions are favorable to the maintenance of such surfaces. The type of grass used will depend upon the region, a mixture of 80 per cent Kentucky blue grass and 20 per cent red top serving fairly well in the northern half of the country while various combinations of Kentucky blue grass and Bermuda grass have been found to prove more satisfactory in the South.

Some sort of paved runway is required for all except the small Class 1 airports. Decision as to the type of paving to be employed must be deter-

mined on the basis of subgrade conditions, load requirements, degree of permanency required, availability of materials and equipment, current and anticipated traffic, meteorological factors such as temperature and precipitation, and available funds.

In general, the paving requirements can be met with the normal types of highway paving. However, paving types that require the kneading and compaction of traffic to maintain their density and life are not suitable for runways. To compensate for the compaction of frequent traffic, softer asphalt and somewhat richer mixtures combined with dense aggregates are necessary to resist the deterioration effect of weather and climate.

Types of hard surfacing that have been used successfully on runways include: sand-clay, gravel, oyster shell, soil stabilized with bituminous oils or with cement, macadam, sand-asphalt, lime-rock, concrete, asphaltic concrete, and bituminous surface treatment. Recent developments in the use of stabilized bases for highway paving indicate that this form of treatment has possibilities for economy in airport paving.

Rough non-skid wearing surfaces are to be avoided because of the excessive tire wear they impose on landing aircraft. For concrete surfaces, those approaching a smooth troweled finish are most desirable. Bituminous surfaces may be finished with a sand seal.

The color and light-reflecting qualities of runways are important in night flying. Surface textures of a light color and those having a high reflecting character are preferable.

Crowns and transverse gradients should be sufficient to expedite surface water runoff, especially in states where freezing occurs. Crowns recommended by the CAA range from 9 in. on runways 100 ft. wide to 18 in. on 200-ft.-wide runways.

All landing strips should have a suitable run-up section at the end of each runway which is kept clear of loose gravel or other loose particles at all times.

Design load for runways

Loads imposed on runways include both static and dynamic loads. When an airplane is at rest, its gross weight for design purposes is customarily considered as distributed between the two main wheels or groups of wheels even though part of the load may be carried by the tail wheel, hence the static wheel load may be taken as half the gross weight of the plane. The range of these loads for the several classes of airports is given in Table I.

There is now no complete agreement as to the magnitude of dynamic load imposed on pavement, but the Civil Aeronautics Authority recommends a factor of two as a design impact factor to be applied to the gross weight of the plane. The apparent discrepancy which appears in Table I between the impact factor of 2 which is recommended to be applied to the gross weight of the plane and the impact factor of 1¼ to 1½ which is recommended to be applied to the unit load on paving is a direct result of the deflection of the tire itself under landing shock.

Regional Offices
CIVIL AERONAUTICS AUTHORITY

P. O. Box 449, Newark, New Jersey
P. O. Box 4327, Atlanta, Georgia
1203 New Postoffice Bldg., Chicago, Ill.
P. O. Box 1689, Fort Worth, Texas
City Hall Bldg., Kansas City, Mo.
P. O. Box 1010, Santa Monica, Calif.
King's County Airport, Seattle, Wash.

Military operations

The figures given by the Civil Aeronautics Authority in the accompanying tables are based on present and anticipated commercial requirements as to size of airplanes. These, it is believed, will be determined by economic rather than structural limitations. Where military use is anticipated, the commercial requirements may well be exceeded, as the size of military planes is not limited by economic considerations. Consequently, airports that may be used for military purposes should be designed in cooperation with the military authorities.

Buildings

The minimum requirement in the way of buildings is a hangar that will accommodate the small planes that are expected to use a Class 1 airport. The higher type airports require an administration building in addition to larger hangars, and all airports require large auto-parking areas. The size of the administration building will depend on the facilities to be accommodated. At the major terminals, in addition to facilities for the accommodation of a considerable number of passengers and for the sale of tickets, there must be space for express, for the post office department, the U. S. Weather Bureau, the airway communications office, and the airway traffic control. There also must be an airport traffic control tower.

Lighting

Provision for lighting, especially floodlighting of the landing areas, is a major element of the airport layout. It is important that the distribution of floodlighting over landing areas be free from abrupt changes in intensity and from shadowed areas. By preparing profiles with lines radiating from the proposed floodlight locations, it can be determined whether or not the beams from floodlights will reach every part of the landing surface. The design must include an underground duct system supplying power to the floodlights, markers and boundary lights. Ample lighting of aprons, loading areas, taxiways and parking fields also must be included in the design.

AVAILABLE AIRPORT INFORMATION

Most of the information contained in this article is from "Airport Design Information" prepared by the Airport Section, Civil Aeronautics Authority, for the guidance of its regional engineers. The airport section also has available bulletins on recommended types of administration and terminal buildings and hangars, standard types of runway drainage, airport lighting and marking, seaplane facilities, and pamphlets on legal questions such as zoning for the protection of airports from obstructions that would reduce their usefulness.

Sketches showing recommended details of concrete airport runway designs are available from the Portland Cement Association, 33 W. Grand Ave., Chicago. The Asphalt Institute, 801 Second Ave., New York City, has published a pamphlet "Asphalt for Airports"; there is a section on airports in "Asphalt Pocket Reference for Highway Engineers," and there is a group of papers on airports in *Proceedings*, Ninth Annual Asphalt Paving Conference, both published by the Institute. The Clay Products Association, 111 W. Washington St., Chicago, has a pamphlet entitled "Airport Drainage."

Information on the stabilization of soils for sub-bases of runways can be found in "Soil Cement Roads," Portland Cement Assn., and in "Soil Stabilization with Asphalt," published by the Asphalt Institute.

Aluminum Dust Combats Silicosis

Contents in Brief — McIntyre Porcupine Mines in Canada makes regular use of aluminum dust dispersed by the face blast to eliminate silicosis hazards. Extensive laboratory and clinical study shows that the dangerous fine silica particles are blanketed if the aluminum amounts to 1 per cent of the rock dust, and if it is sufficiently fine and grease-free. Treatment of silicotic cases with aluminum dust has also shown promising results.

As the outcome of investigations begun in 1932 to discover means of preventing silicosis in mining and tunneling, McIntyre Porcupine Mines of Canada is making regular use of aluminum powder. A small amount of powder placed at the face of the workings after loading a round is dispersed by the shot, and its effect is to blanket the dangerous fine particles of silica and protect those who must breathe the dusty air against contracting silicosis.

R. J. Ennis, general manager of the mines, states in the annual report of the company for the year ending March 31, 1940, that "The dispersal of minute quantities of metallic aluminum powder in certain parts of the mine to eliminate the hazard of silicosis continues with satisfactory results. With the endorsement of the Ontario Board of Health and the Ontario Compensation Board, a clinic has been established at St. Mary's Hospital, Timmins, Ontario, for the treatment of silicotic miners with aluminum powder. Dr. D. W. Crombie, Medical Superintendent of the Queen Alexandra Sanatorium, London, Canada, is directing the clinic."

As indicated by Mr. Ennis' statement, use is also being made of aluminum dust to treat established cases of silicosis. Results of such treatment have not been published, but private reports indicate that a number of silicotics whose bad lung condition was definitely determined by radiograph were greatly improved by the treatment, the disease appearing to be arrested. The possibilities of treatment are considered promising.

Research into the preventive effect of aluminum dust was outlined in two papers by the late J. J. Denny, metallurgical engineer, and W. D. Robson, chief surgeon of the company, and Prof. Dudley A. Irwin of the University of Toronto, published in the Canadian Medical Association Journal in 1937 and 1939. The research began in 1932 with a study of the effect of mine gases, hydrogen ion concentration and addition of various substances to siliceous rock dust, on the susceptibility of animals to silicosis. It was found that the addition of certain compounds to the silica dust reduced its solubility in laboratory tests but had no beneficial effect when inhaled. Early in 1936, however, Denny and Robson discovered at the McIntyre mines that "the presence of small amounts of metallic aluminum almost completely prevented siliceous material from passing into solution." Animal experimentation followed at once, and tests on comparison groups of rabbits in atmospheres of quartz dust, and quartz, plus 1 per cent of metallic aluminum, showed that silicosis was inhibited by the aluminum admixture, even during many months' exposure to the dust.

An important observation made at the mine was that men working in the crusher houses do not develop silicosis although the mill feed contains about 35 per cent of free silica. Check experiments on animals confirmed the observation, no fibrosis developing in animals exposed to the dust up to a period of 12 months. The "antidotal rocks" present in the mill feed, when added in equal proportion to quartz, reduced the laboratory solubility of the quartz by 70 to 80 per cent. Further study suggested that an aluminum hydroxide film formed by these materials on the quartz was responsible for the preventive effect. Study of the effect of aluminum powder followed.

In actual trials in the mine, small quantities of aluminum powder, varying from 3 to 20 grams, were dispersed in the dust cloud produced by the face shots. Tests of the dust samples taken 150 to 300 ft. from the face shortly after the blast showed a reduction in solubility of the quartz of 90 to 98 per cent as the result of using the aluminum powder.

Conclusions reported in the 1939 paper are: No animals whose lungs, on analysis, contained 1 per cent or more of metallic aluminum showed any evidence of silicosis up to periods of 17½ months, in contrast to well developed silicosis in control rabbits exposed to quartz dust alone in seven months. Silicosis may also be prevented by inhaling aluminum dust daily independently of the siliceous dust. Inhaling aluminum dust in large quantities over long periods of time showed no effect on the general health of the animals and no evidence of toxicity or damage to tissues. Aluminum dust for the prevention of silicosis should be of a particle size below 5 microns, should be grease-free, should be uniformly mixed in the inhaled dust and should bear a definite percentage to the dust (1 per cent or more).

As to explosion hazard the investigation showed that aluminum dust in the concentration necessary to prevent silicosis is hundreds of times below the explosive concentration of aluminum powder.

Associated studies in the physical laboratory indicate that the aluminum forms an extremely fine film on the silica particles. Special investigations made in the Bell Telephone Laboratories, New York, and reported by L. H. Germer and K. H. Storks in "Identification of Aluminum Hydrate Films of Importance in Silicosis Prevention," *Industrial and Engineering Chemistry*, Nov. 15, 1939, showed that the protective film is a hydrated oxide of aluminum.

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Denver Decreases Its Water Loss

D. D. Gross, chief engineer of the Denver, Colo., municipal water works recently reported that at the present time raw water diverted from natural sources into the city's system was being used by the consumers to the extent of 90 per cent. In 1926 over 1 gallon in each 5 was lost through leakage and evaporation. In all a saving of over 3 billions of gallons was realized in 1939 as over 1926.

FIG. I. HERE AT CORAM, A 9.6-MILE BELT DELIVERS AGGREGATE TO A HOPPER FEEDING THE BELT TO THE DAM.

Shasta Dam Construction Layout

Contents in Brief—Materials are brought to the mixer bins by belt; concrete moves out in two-compartment hopper cars, via a circular track whence it can be discharged into 8-cu. yd. buckets on a loading dock under seven radial cableways. The cableways, which reach out over the entire dam and powerhouse area, center at the top of a 460-ft. steel tower, each with a gudgeon pin anchorage articulated by four roller bearings. Connections are designed to take cable pull up to 600,000 lb. without risk of "freezing."

"NOT A RECORD FOR SPEED but a world's record for the lowest cost concrete placed according to specifications" was the objective in laying out the plant that will mix and place concrete in Shasta Dam on the Sacramento River 12 miles above Redding, Calif. Since the contract held by Pacific Constructors, Inc., calls for placement of 5,700,000 cu. yd. of concrete, it is good economy to take advantage of all possible savings in handling costs. This has introduced such novelties as the seven concentric cableways, a tower whose steel frame is taller than the ultimate height of the dam itself and a circular track at the head tower for delivery to the cableways. An unusual feature is the special care taken to prevent any risk of setting up fatigue strains in the tower-top cable anchorages. Essentials of the construction layout, set forth in the following, supplement an earlier account of this major item in California's Central Valley Project published in *Engineering News-Record*, Nov. 9, 1939, p. 612.

Starting point in the layout of the construction plant was the station at Coram on the Southern Pacific railroad a mile below the damsite. Coram was selected by the government as the point where concrete aggregates would be delivered to the contractor. To save the risk of a river crossing, the contractor preferred to put the concrete mixing plant on the Coram side of the river and, naturally, at a level high enough so that concrete delivery could be by gravity.

These considerations located the mixing plant high up on the right bank of the river and to this point aggregate is delivered by belt conveyor and cement comes via pipe under air pressure from the blending silos. Between Coram and the mixing plant, a location was found for aggregate storage piles and cement blending plant (Fig. 2).

As to the best means for moving concrete from mixing plant to dam and powerhouse (an area some 3,500 ft. long with a maximum width of 1,050 ft. from the axis of the dam to the west corner of the powerhouse), Frank T. Crowe, general superintendent for the contractor, worked out an arrangement of radial cableways born of confidence arising from long experience with them, most recently in building Boulder and Parker dams.

The objective in making any cableway layout for concrete delivery,

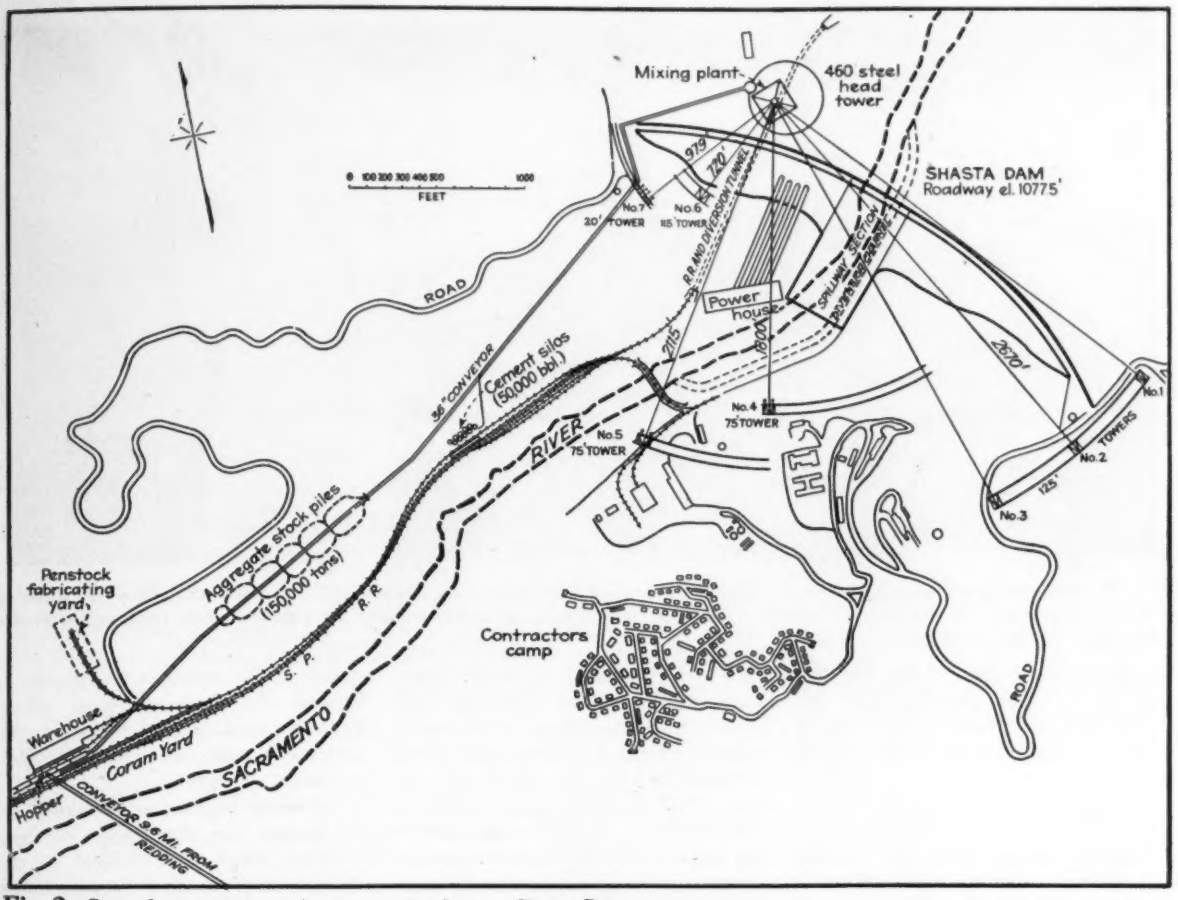

Fig. 2. General arrangement of contractor's plant at Shasta Dam.

Mr. Crowe points out, is complete coverage of the area to be served, placing the mixing plant as near as possible to the center of mass. To accomplish this at Shasta Dam, it was most economical to use the radial cableways mentioned above, centering in a structural steel head tower. The latter would be expensive because (a) it would be more than 560 ft. high (460 ft. above ground surface) and (b) it would have to be stiff enough so that its top could take cableways pulls up to 600,000 lb. Despite the costly tower and the need for pioneering design in the tower-top cable fastenings (to be discussed later) this arrangement was adopted. The tower contains 2,576 tons of structural steel and its square base measures 185 ft. on a side.

Construction of such a tower was a major undertaking. Single pieces weighing as much as 42 tons were brought to the site ready to be hoisted in place by a pair of creeper cranes that worked up the tower sides as the steel was erected.

The major problem, however, was at the top of the tower where tension and torque of enormous proportions had to be handled. Heretofore, Mr. Crowe points out, the problem of how to obtain complete flexibility so as to avoid danger of crystallization at cableway anchorages, has been only partially solved. Serious wrecks have occurred because lubrication of a joint could not be effected, as the design contemplated, and the result has been crystallization and breakage. It was unthinkable that any such risk as this could be permitted at the top of a high tower and hence a new type of cable fastening had to be developed—nothing less than roller bearings to insure free articulation in vertical as well as horizontal planes on each of the main cableway connections at the tower top could be used.

This was accomplished by using a heavy, solid gudgeon pin designed with four roller bearings, two to give the pin motion about a vertical axis, the third acting as a thrust bearing to take the vertical component and a fourth around the pin holding the draw bars leading to the cable. This design (Fig. 5) provides a completely articulated fastening despite the heavy loading. The heavy gudgeon pin bearings of special design were furnished by the Fafnir Bearing Co., New Britain, Conn. The assembly of 5-ft. sheaves and cheeks slung from each gudgeon pin weighs 15 tons.

The seven main cableways are 3-in. track cables with operating equipment (at the 260-ft. level in the tower) designed to give lifting speeds with 2-ton working loads, (8-cu. yd. buckets) of 300 ft. per minute and carriage speeds of 1,200 ft. per minute.

The concrete mixing plant, a steel structure standing at one corner of the head tower, is designed for a capacity of 6,000 cu. yd. per day and has automatic batching equipment and five 4-cu. yd. mixers. Four of these discharge into a common hopper; the fifth discharges into a separate compartment within the common hopper where the arrangement is such that the concrete can be passed on into the main hopper or diverted to a chute provided for grout or powerhouse concrete. The 85,000 cu. yd. of concrete for the

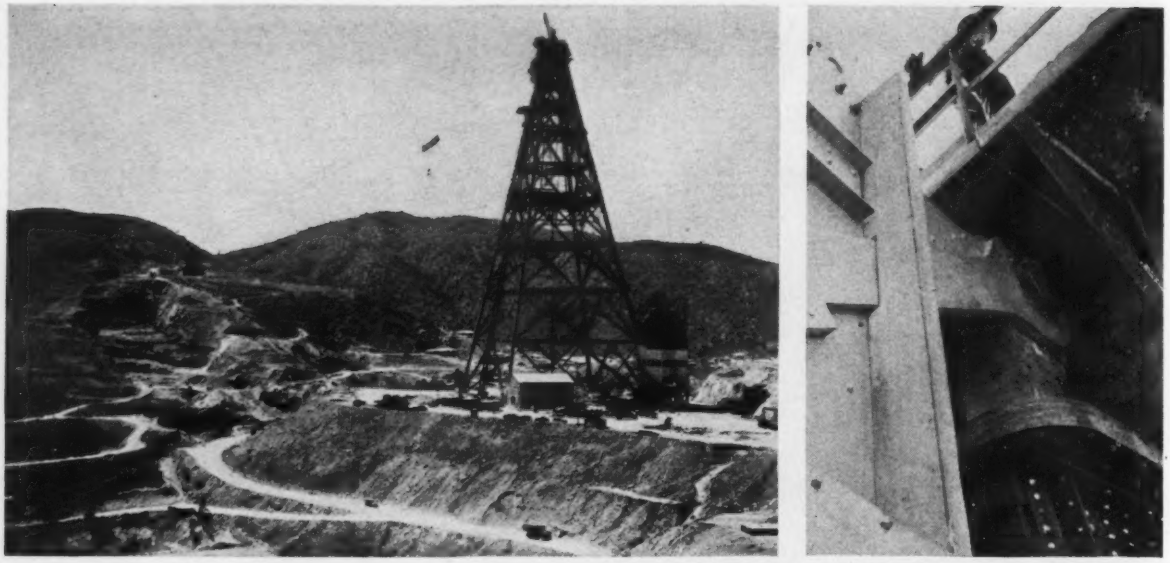

Fig. 3. Cableway head tower and concrete mixing plant stand high up on the right abutment. Across the valley are three of the six tail towers. At the right is shown a detail of the gudgeon pins that transfer the pull from 3-in. track cables to the steel frame of the head tower.

powerhouse is richer, wetter and has smaller maximum size aggregate because of the necessity for placing it around reinforcing in walls, scroll cases, etc.

Intermediate delivery

Getting concrete from the mixers into 8-cu. yd. buckets under all seven cableways called for some form of intermediate delivery to a point far enough from the tower center so that there would be no interference in serving the several lines. This was accomplished by building a circular track on a 210-ft. radius on which standard gage cars move from the charging hopper under the mixing plant to points around the circle convenient for cableway pickup.

Hopper cars for this circular track are job-assembled, all-steel motor cars carrying two 8-cu. yd. hoppers with quick-discharge gates. A third rail around the circular track serves two 125-hp electric railway motors, one on each truck, connected in parallel and controlled from the operator's station on the discharge side of the car. Spring shoes on the trucks extend under a 2x10-in. plank, which is mounted over the third rail for safety, bringing a constant supply of 300-volt d.c. current for the motors and for a compressor which supplies air pressure for operating brakes and the discharge gates. Two butane tanks, such as are ordinarily used on motor trucks, make convenient and inexpensive air receivers. The circular track is laid with 90-lb. T-rail and the third rail is a 40-lb. section.

Because the hopper cars (there will be two or three when pouring reaches peak) run on a circular track, the wheels on the inside rail are of smaller diameter by ⅜ in. than those on the outer rail, thus eliminating frictional slippage. The air-operated gate mechanism is so designed that at a turn of the air control valve the gate snaps open to its full width so as to give almost instantaneous release to the 8-cu. yd. charge. The operator's platform is between the discharge gates where there is unobstructed view of the cableway buckets, an important factor in speedy delivery.

Cableway buckets to be loaded by the hopper car, stand on a landing platform below and outside of the circular track so that the discharge chute on the hopper car will be just above the bucket top. This bucket landing or dock is a bench cut on the arc of a circle. The vertical face is a fender pier built to serve as a bumper against which incoming buckets can strike in making their landings. The fender pier is built up of vertical, 12x12-in. timbers on 2-ft. centers, held by 1-in. tierods running back to concrete anchors in the fill behind them. These uprights are set in a concrete base; they are lagged on the inside to prevent the fill from working out and on the outside to provide a wearing face for the fender. The lagging consists of 3x12's placed horizontally.

A new feature in the 8-cu. yd. buckets for this job is the use of a sheave within the bucket around which to pass the line that holds the gate closed. Instead of the old design in which the gate is pulled directly upward to effect closure, the introduction of the sheave pulls gate and bucket toward each other with equal effort in both directions, thus constituting a pincer action.

The concrete delivery program, still subject to adjustment, is now

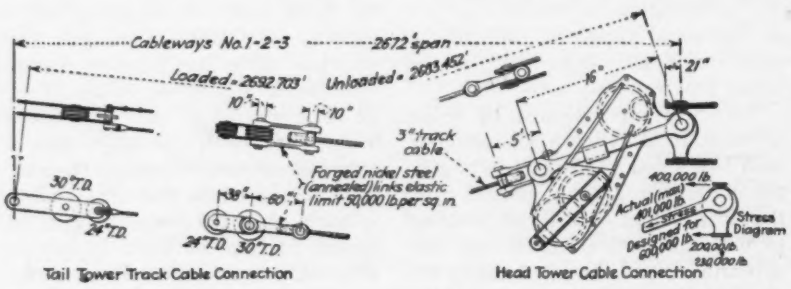

Fig. 4. Connections for the 3-in. track cables used for the cableway system.

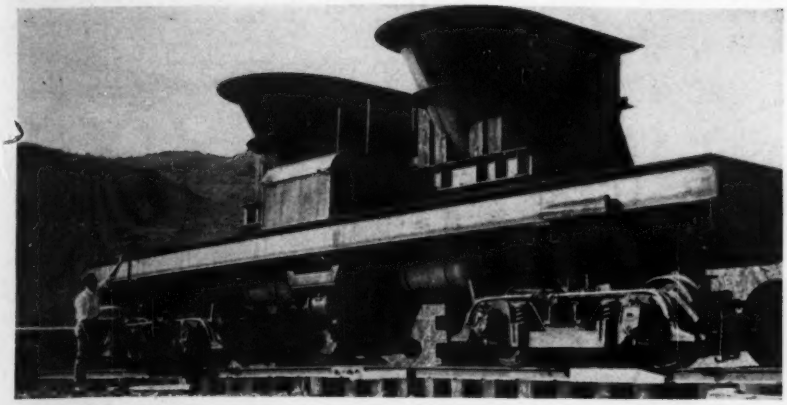

Fig. 6. Cars operating on the circular track have two 8-cu. yd. hoppers.

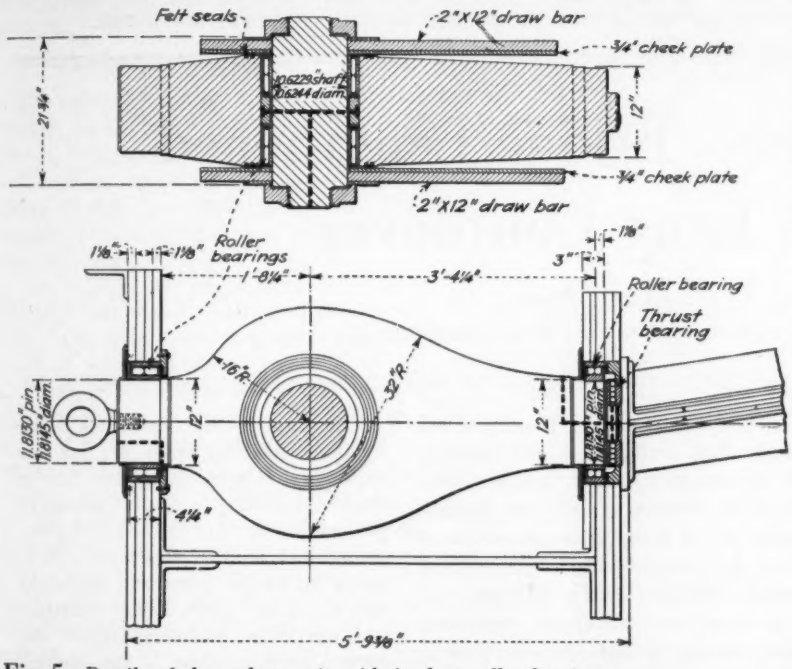

Fig. 5. Details of the gudgeon pin with its four roller bearings.

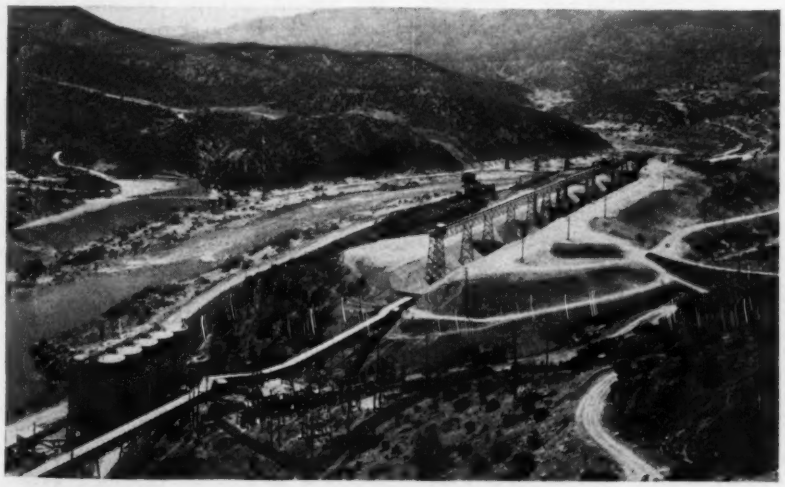

Fig. 7. Stockpile, cement silos and covered belt to the dam.

laid out on the basis of 20,000 cu. yd. for the first month, (start of pouring was scheduled for June 15), 90,000 cu. yd. in the second month and thereafter, for the most part, 180,000 cu. yd. per month. Final concrete is to go in late in the summer of 1943. This schedule gives ample margin, inasmuch as under the specifications only 1,300,000 cu. yd. of concrete can be placed before the railroad is taken out of the canyon and rerouted over the relocated line now under construction, an event now scheduled for September 1, 1941. This concrete yardage could be put in during the 14 months following July 1, 1940, by pouring an average of less than 93,000 cu. yd. per month.

Exact yardage of concrete to be placed will not be known until all the excavation has been completed, bedrock entirely uncovered and surveyed. Communications between operators of cableways and signal men on the dam will be by telephone over a carrier system using radio frequencies, and by direct wire bell signals.

Shasta Dam is being built by the U. S. Bureau of Reclamation, John C. Page, commissioner; R. F. Walter, chief engineer; Walker R. Young, supervising engineer of the Central Valley Project, and Ralph Lowry, construction engineer, Shasta Division. Grant Bloodgood is resident engineer at the dam.

The contract for concrete placement was let to Pacific Constructors, Inc., for $36,000,000, not including cement, steel, sand and aggregate, which are all supplied by the government. Officials of this firm include W. A. Johnson, president; Frank T. Crowe, general superintendent; B. W. Goodenough, chief engineer, and L. P. Sowles, plant engineer.

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Utilities Most Commonly Owned By Cities

Waterworks, sewage disposal plants, cemeteries, airports and auditoriums are the most commonly owned public utilities in cities of the United States, according to a survey in the *1940 Year Book* of the International City Managers' Association.

Nearly 73 per cent of the cities over 5,000 population own their waterworks. The number of municipally-owned sewage disposal plants increased 3 per cent last year.

FIG. 1. TRAIN OPERATING OVER PART OF NEW BRIDGE AS STEEL ERECTION WORK CONTINUES ON REMAINDER.

Simple Truss Bridge Lengthened Into Long Cantilever

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Contents in Brief—Widening the Atchafalaya River for flood control required lengthening the Missouri Pacific R.R. bridge at Krotz Springs, La. Scheme was to separate two of the existing 300-ft. simple truss spans by 720 ft. of new steel in the form of twin cantilevers having a common anchor arm, the 300-ft. spans being hung from the ends of the new cantilevers. The 300-ft. span to be removed to the far end of the new bridge was rolled out of position simultaneously with the roll in of 300 ft. of new steel, so as not to interfere with train operation. While remainder of new steel was being erected trains operated over a detour trestle. Work involved removal of two old piers and the construction of two new deep caisson piers of special design, incorporating a jet system and sounding wells in the walls and three dredging wells.

PRELIMINARY TO WIDENING the Atchafalaya River as a part of the federal flood control project on the Lower Mississippi, the four-span single track Missouri Pacific R. R. bridge over the Atchafalaya at Krotz Springs, La., has been lengthened by the process of building into the bridge a new 721-ft. cantilever structure, and suspending two of the existing 300-ft. simple spans from the ends of the new cantilever arms. This unusual transformation required sinking two deep open caisson piers of interesting design and rolling out one of the existing simple spans for relocation at the far end of the new steelwork together with the simultaneous rolling in of a like length of new cantilever structure, all without interrupting railway traffic.

The original railway bridge at this site, consisting of a 300-ft. swing span flanked by two 300-ft. simple spans, was constructed in 1907, and shortly thereafter was extended on the west by the addition of an identical 300-ft. simple span. This was necessary due to erosion on the west bank of the river. This structure served until a 1936 amendment to the Federal flood control act of 1928 provided authority for doubling the channel capacity of the Atchafalaya River and authorized federal expenditures for any necessary bridge extensions.

Comparative cost estimates made by the consulting engineers for two methods of lengthening the bridge showed a saving in cost of about 15 per cent in favor of the adopted plan, as against adding two new 300-ft. spans on deeper piers and replacing one of the old piers. Fewer substructure operations and considerably less hazard were involved in the adopted plan (Fig. 2), which contemplated the use of a twin cantilever with common anchor arm, to the cantilever ends of which two of the old 300-ft. spans were to be attached.

In detail, the plan required that the principal reactions were to be carried by two new caisson piers, while a new pile pier would support the westerly end of one of the 300-ft. spans after it was shifted to the west end of the new steelwork. Panel lengths of the new steelwork were so arranged that a 300-ft. length of the new steelwork could be erected alongside the old 300-ft. span which was to be shifted, and rolled into place while the old span was rolled out. The additional steel of the new structure was then to be extended westward from this rolled-in portion over the proposed new channel, while rail-

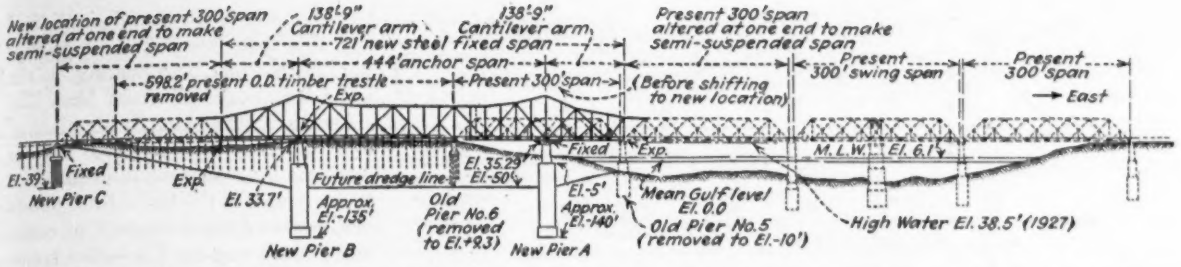


Fig. 2. Plan of reconstruction of Krotz Springs bridge showing how an existing 300-ft. span was moved 720 ft. and utilized at the end of the new steelwork that filled in the gap.

way traffic would be detoured from the new construction through an opening to be left in the north anchor arm truss until the time of final closure.

The piers required

The two deep cellular caisson piers (shown in Fig. 3) are of a unique design, employing triple dredging wells which extend upward into the shafts of the piers, thus saving the expense of a cofferdam and permitting the weight of the shaft concrete to be utilized in overcoming skin friction while the caisson is sinking. The three wells were also used, instead of the single-well circular design that is common in this southern vicinity, to permit the contractor to dredge in the outer wells without interference from the trusses of the existing bridge spans. As a further aid in maintaining the caisson in position during sinking, jet systems were designed to operate in quadrants, and closely spaced 8-in. dia. sounding wells were provided in all caisson walls. Due to the hollow cellular construction of these piers, the pressure imposed on the foundation strata was only 4 tons per sq.ft greater than that imposed by the weight of existing earth.

How the piers were sunk

Pier construction operations were begun at the site of new caisson Pier A, located on the west bank of the river, the contractor utilizing a stiffleg derrick placed on a platform at El. + 27 alongside the bridge from which the derrick worked with flat boom under the steelwork. The placing of concrete lifts was alternated with open dredging, until the caisson cutting edges had sunk to El. −60, when a list of the top of the caisson was noted toward the river. Some correction of this list was accomplished at this point by jetting through the

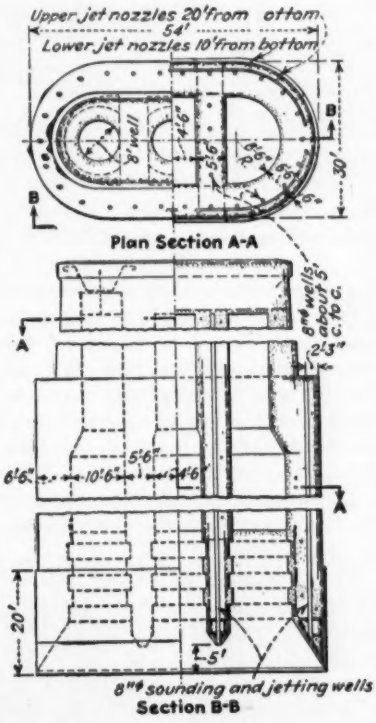


Fig. 3. Details of new deep caisson piers notable for their jetting systems and three dredging wells.

sounding wells on the bank side of the caisson.

When Pier A caisson had reached a cutting edge elevation of −77, the list had reached 9½ in., and a shoreward cable pull was applied, augmented by air jetting through the northwest and southwest quadrants, until the list was entirely removed during the next 20 ft. of sinking. Thereafter the pier sank vertically, following closely upon the dredging, until its shaft had been poured and the cutting edge had reached the plan elevation of −140, in a formation of coarse sand and gravel. The bottom seal was placed by tremie.

Operations on Pier B, located on high ground to the west, had meanwhile been carried on intermittently with Pier A operations. The detour embankment and trestle had previously been completed, and railway traffic was being run over this detour free of Pier B operations. Some of the initial work on Pier B was slow because of the necessity for clearing the pier site of old piles from the former mainline trestle.

A tendency presently developed in Pier B caisson to tilt toward the west.

Fig. 4. Ready to roll out the 300 ft. truss span and roll in the new steelwork, which is part of a long span cantilever.

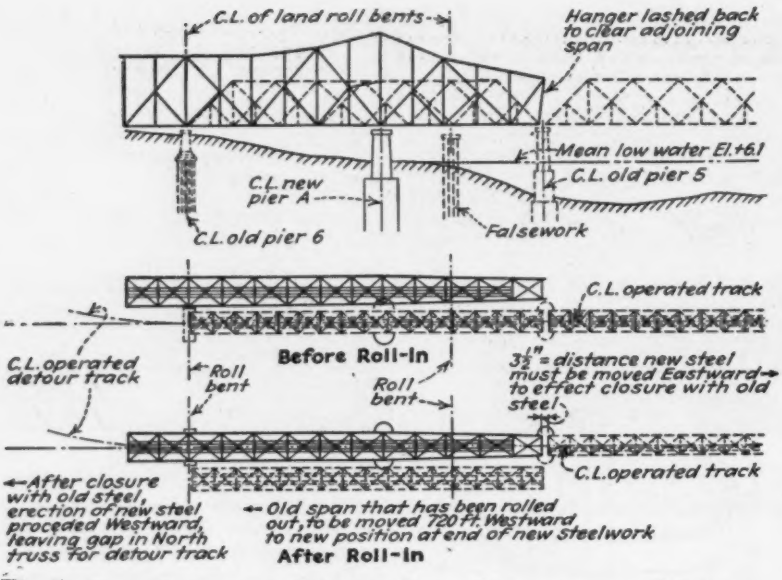

C.L. of land roll bents
Hanger lashed back to clear adjoining span
Mean low water El.+6.1
C.L. old pier 5
C.L. new pier A
C.L. old pier 6
Falsework
C.L. operated track
Before Roll-In
C.L. operated detour track
Roll bent
Roll bent
3½" = distance new steel must be moved Eastward to effect closure with old steel
C.L. operated track
←After closure with old steel, erection of new steel proceded Westward, leaving gap in North truss for detour track
← Old span that has been rolled out, to be moved 720 ft. Westward to new position at end of new steelwork
After Roll-In

Fig. 5. How old truss was rolled out and part of new span was rolled in.

Accordingly air was turned into the east jetting quadrants, and timber kickers were applied on the west, by which means this caisson was maintained plumb throughout the sinking.

When the cutting edge of Pier B caisson had reached El. −100, considerable skin friction was retarding the sinking, and dredging had to be carried below the interior cutting edges to cause movement. The caisson was now in about 50 ft. of coarse sand, overlain by blue silty clay. Very little seepage of water into the wells occurred, and water was run into them from the top during the sinking. The shaft concrete was placed early to gain weight for the sinking, and in addition 22 ft. of sand, amounting to 350 tons, was placed inside the forms above these shafts; yet the caisson "froze" tight with its cutting edge at El. −135.5, about 4½ ft. above the desired elevation. At this point, the buoyant weight of pier concrete plus the superimposed sand load was exerting a force to overcome skin friction amounting to 620 lb. per sq.ft. of external caisson surface. A 3-in. diameter jet was now worked outside the pier on the north side, and dredging was carried to 6 ft. below the cutting edges at well centers in an effort to resume sinking, but a blow-in occurred causing sand to rise 5 or 6 ft. inside the caisson and the effort had to be abandoned. The caisson was, therefore, sealed above plan elevation in good coarse sand and gravel.

The possible accuracy of positioning deep caissons of this design may be seen from the fact that the lower caisson of Pier A was only 9¾ in. out of plan position, and that of Pier B only 6 in. out of position. Since eccentricities in excess of this had been allowed for in the design, the shafts were built off center of the lower caissons by those amounts, and the bridge seats are, therefore, in the correct positions.

First operation on the superstructure contract was to construct timber pile falsework bents for the support of the new steelwork that was to be erected (prior to rolling it into place) in a position north of the most westerly old 300-ft. span (Fig. 2). Two principal roll bents were established, one opposite old pier 6 and the other opposite the far quarter point of the old 300-ft. span. It was upon runways mounted on these two bents that the rolling out of the old steel and rolling in of the new was accomplished.

The various motions of the roll-in are shown diagrammatically in Fig. 5. The time available to the contractor for this operation was the 9-hr. interval between the last morning train at 4 A.M. and the first afternoon train at 1 P.M., and careful preparations had been made to carry out the move as expeditiously as possible. Railroad forces were on hand to break track and remove a portion of the adjoining old trestle as soon as the last morning train had crossed. The shore end of the old 300-ft. span had already been set up on roll points on a transverse runway laid over old pier 6, and now this span was jacked off its support on old pier 5 to bearing on the roll beams at the river roll bent. The old and new steel being firmly blocked together, they were made to roll as one unit by means of lines attached to the trusses at the two rolling points and operated by a locomotive crane and a shore engine. The entire transverse roll was accomplished in half an hour and was completed by 7:30 A.M.

It was now necessary to install the floor stringers in the end panel next to the adjoining 300-ft. span, and to roll the new steel 3½ in. longitudinally riverward to seat these stringers and close the connection between the old and new steel. To jack the new steel on and off the rollers for this last longitudinal move, to complete the hanger and floor steel connections and to jack the new steel to the required elevation for its position in the final structure were time consuming operations; but they were accomplished successfully with but slight delay to the afternoon train.

This completed the most difficult phase of the superstructure erection. During the next month, the 300-ft. span that had been rolled out of the old structure was mounted on longitudinal roll beams and moved 720 ft. westward, and then transversely northward to its final position. Erection of the remaining new steel was meanwhile taking place, and as a last operation those members where the detour track passed through the north truss (Fig. 1) were erected.

Personnel

Lt.-Col. Wm. F. Tompkins was district engineer of the Second New Orleans District, Corps of Engineers, during the progress of these bridge alterations. C. S. Kirkpatrick is chief engineer and R. Owen construction engineer of the Gulf Coast Lines, Missouri Pacific Railway. Otto F. Sorgenfrei was resident engineer for the firm of Modjeski and Masters and the J. F. Coleman Engineering Co., who were associated as engineers on the work and prepared plans for and supervised construction of the bridge alterations. Paul H. Galbraith was superintendent for the general contractor, Missouri Valley Bridge and Iron Co., whose bid for the work was $723,779. The superstructure work was handled by the Mt. Vernon Bridge Co. on a subcontract.

Paving the Pennsylvania Turnpike

Contents in Brief—Eight Pennsylvania contractors and thirteen from nine other states concentrated what probably is the most modern paving equipment ever assembled for the recent drive to pave more than 140 miles, four 12-ft. lanes wide, of the Pennsylvania Turnpike. Equipment and methods are similar on most of the contracts; some new equipment has appeared; hauling instead of pumping water is general practice. PWA deadline has made rush work necessary and all outfits are working at least two shifts per day.

NEAREST APPROACH to the "Futurama" highway of the New York World's Fair is being rushed to completion across the mountains of Pennsylvania as the Pennsylvania Turnpike Commission attempts to meet the PWA deadline on a 160-mile superhighway project requiring two 12-ft. lanes of 9-in. concrete in each direction with 10-ft. shoulders and 10-ft. planted separating strip, in less than 21 months from the time funds were available for making plans. The Turnpike required removal of 25 million cubic yards of earth and rock excavation in open cut, construction of seven tunnels aggregating nearly seven miles in length and construction of 307 drainage and grade separation structures other than culverts. Nominal right-of-way width is 200 ft., graded roadway 78 ft., maximum grade 3 per cent, and maximum curve 6 degrees (for most of the roadway length 4 degrees is the maximum, sharper curves being confined to one 6-mile section).

First contracts for paving were awarded in late July and August of 1939 and included grading and structures through the relatively shallow grading depths of the eastern end of the turnpike. Grading, where cuts and fills were considerable, was let separately, fills generally being permitted to settle through winter before paving was laid. Last paving contracts were awarded March 13 making a total of 153 miles exclusive of the seven miles included in the tunnel contracts. Only the equivalent of 13 miles of 4-lane pavement were laid last fall leaving 140 miles to be placed between April 1 and the June 22 contract requirement.

Pavement is a simple 9-in. flat slab placed 12 ft. wide, or 24 ft. with a center false joint (some contracts required placing in 12-ft. width only). Superelevation is used at almost all curves and spiral easements on curves sharper than 2 degrees but the 12-ft. width of lane is maintained so that uninterrupted machine finishing can be carried on. No curb is used on any section, drainage being cared for off the paved area.

Fig. 1. A large 34E paver followed by a 27E paver on the County Construction Co. contract. Two spreaders worked with the paver, one on the 7-in. lower course and one on the 2-in. top course. These are followed by two finishers and a float.

Most notable feature of the paving procedure is the similarity of equipment and methods despite assembly of machinery and personnel from a total of eleven states. Nearly all are familiar to highway builders throughout the country though two new machines are in use.

Materials

There is no acceptable fine aggregate in the area crossed by the Turnpike. For the eastern end of the project sand is secured largely from near Baltimore, Md., and Williamsport, Pa., and for the west end from the Pittsburgh area. Coarse aggregates, ¼ in. to 1 in. and 1 in. to 2 in., kept separate, are limestone from local quarries along the route except some gravel used on the westerly contracts.

Cement and sand made its initial move by rail in nearly all cases, coarse aggregates only in a few instances. Contractors' batching plants were sometimes set up on the rail sidings, in others near the Turnpike with truck delivery of cement and sand from the rail head and stone direct from the quarries. All cement is delivered in bulk in hopper bottom dump cars. Cars generally are unloaded by screw conveyor undertrack to the boot of a bucket elevator taking the cement to a steel portable bin averaging 300 bbl. capacity from which the cement is batched or trucked to another bin at the batcher plant set up. Cement hauling units are generally dump body trucks with a tight cover as shown on p. 55 delivering to the top of a hillside plant bin or to a bucket elevator at a surface level bin.

Sand must remain in a stockpile not less than 12 hours to equalize the moisture content. Railside plants are, if possible, located to accommodate two stockpiles within crane reach of the bins. Considerable stock was accumulated through the winter on many jobs as limited rail facilities for handling Turnpike requirements and advance payment provisions for stockpiled materials made large quantities on hand an economic desirability for contractors. General equipment was gondola cars and clamshell buckets for unloading and reclaiming. An exception to the rail delivery was a 125-mile one-way truck haul undertaken to replace rail delivery plus an 8-mile truck haul, at same cost to contractor, when sand was freezing in cars and winter shutdown for the trucker was in prospect. Coarse aggregate is delivered by rail to jobs near both terminals of the Turnpike but through the central section delivery is by truck direct from quarry to batch plant with storage piles at either or both points.

Batching plants

Portable batching plants, generally all steel but some with steel frame and wood sides, are used on all jobs. In many locations a "hillside" plant has been practical where provision is made for truck delivered material to be dumped directly into the top of the bin as shown in Fig. 4. Reclamation from stockpiles was by clamshell bucket or truck loader but normal supplies have been trucked in as used wherever possible. Bins used are 100 to 200 tons and from 1 to 4 compartments, some providing for sand, 2 coarse aggregates, and cement through one bin. Some plants are set up for a single "drive through" operation; others require backing under separate hoppers for sand, stone and cement. All batching is by weight as required in the specification.

Dump truck requirements for hauling stone from quarries, sand and cement from rail connections to roadside batch plants, and batches to the pavers constitute a real problem in Turnpike construction. At the height of the construction a serious shortage has developed, not easily remedied as the peak is far too short a time to justify new equipment. Practically all the hauling is done by trucks on a ton-mile or batch-mile basis, little by the contractors' own machines. Operators have an incentive to "ride hard," perhaps even to overload and consequently some of the trucks are breaking down under the 16 to 21-hour daily operation.

Hauling to the paver is done in dump trucks except on one job where conventional train boxes are set on the truck chassis and swung over the skip with a derrick on the mixer. Two batches each weighing about 5,500 lbs. for the 34E mixers, constitute the usual load for medium sized trucks though three batches are sometimes hauled.

New pavers 34E double-drum

All mixing is done by pavers on the roadway. New equipment is 34E double-drum type mixers except one 27E, also double-drum. Several previously used pavers are 27E single-drum but in many cases these are used (1) two together, usually boom to boom (2) supplementing a 34E double-drum, placing the 2-in. top above the reinforcing mesh following

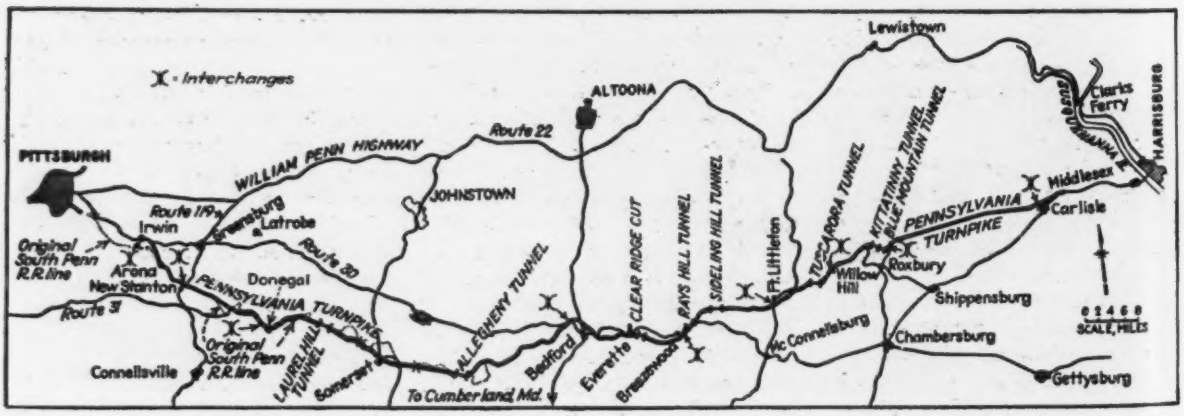
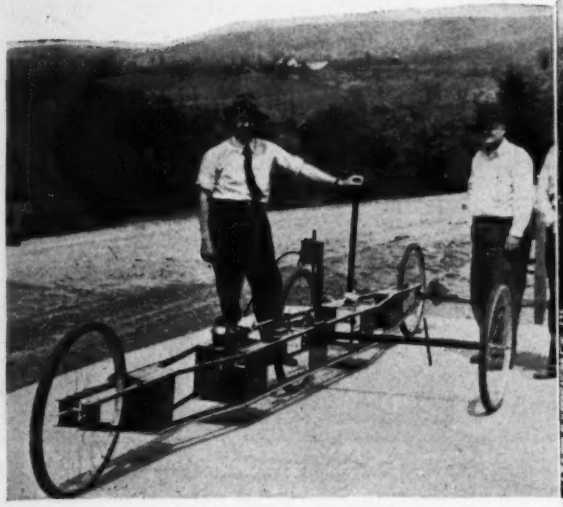
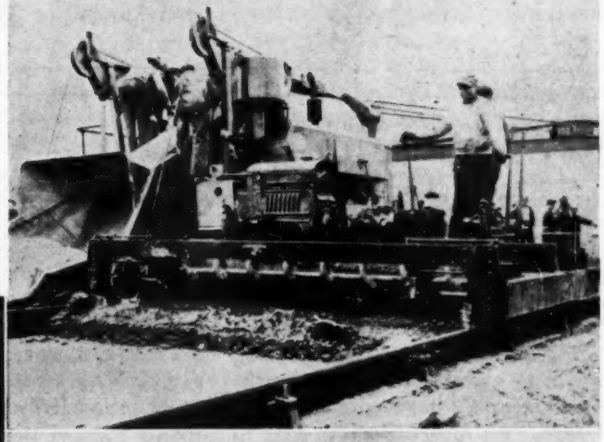


Fig. 2. Route of the Turnpike. Maximum two per cent grades and six degree curves, will speed traffic through the mountains of Pennsylvania.

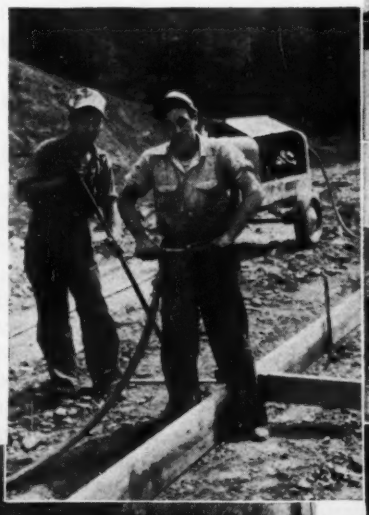
Material deliveries for the Turnpike have taxed the capacity of the railroads.



Sixteen-foot recording profilometer. E. W. Martin, at left, is co-developer and operator.



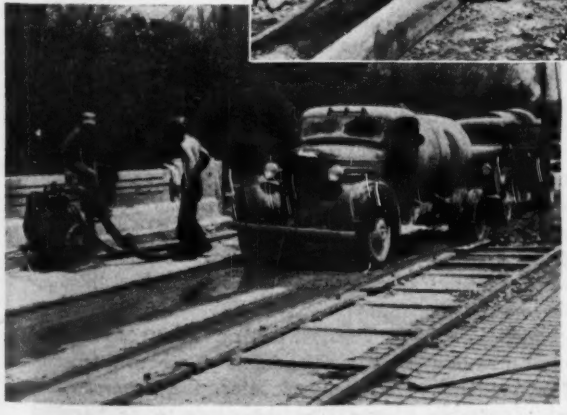
Screw-type spreader on the job of the County Construction Co.



Pneumatic pin drivers are in general use. This is on the Andrews and Andrews job.

Finegrader on Baldwin Bros. job (machine grades an 8-ft. width of roadway).

Steel dump body made tight and covered for handling nine tons of cement.



Pump filling tank trucks for mixer and curing use.

the bottom 7 in. placed by the larger unit. In some cases elaborate sequences have been worked out so that the two mixers place concrete with a minimum of shuttle movement for both paver and spreaders. Schemes of paving are varied. Some contractors are paving the two outside lanes first to permit start of shoulder work. Others are paving inside lanes first to prevent trapping rain water. Still others, after an argument with the engineers, are paving 24 ft. at one time using a cleft type false joint to create a plane of weakness at the 12-ft. line.

Rapid moving units laying upward of 3 miles of 12-ft.-wide pavement per week require several miles of pipeline and frequent changes of long hose connections. Consequently two-thirds of the contractors are hauling water for mixing and curing. Some of those pumping are using trucks to sprinkle for curing as the shoulder or a completed slab permit ready access to the concrete. Five to eight tanks, generally 1,000 to 1,500 gallons capacity, some equipped with small pumps to give pressure to the sprinkler line, service each paving outfit, the number depending on local conditions. Suctions on the pavers take the water from the tank trucks for mixing. Where pumps are used pipelines vary from 3 in. to 2 in. with maximum pumping distances about 3½ miles. Water generally comes from approved surface drainage streams crossed by the Turnpike. Contractors hauling water are enthusiastic about the method, claiming savings in time, money, and headaches.

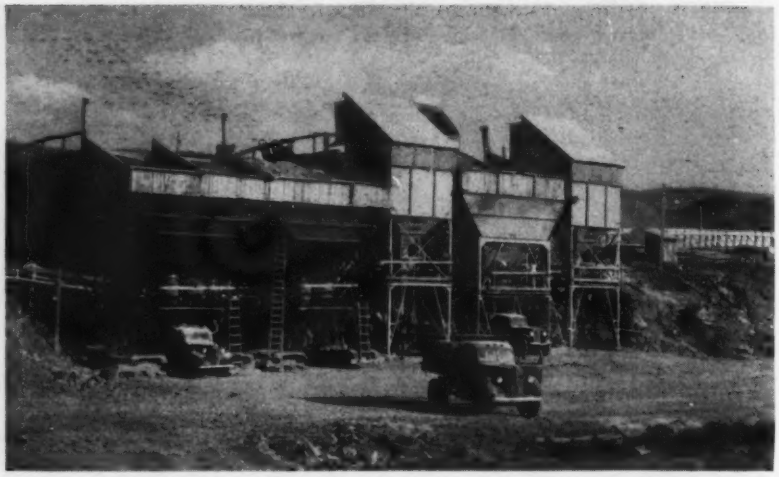


Fig. 4. Union Construction Co. "hillside" plant set up to make 37.4-cu. ft. and 29.7-cu. ft. batches for 34E and 27E pavers. This layout was used to advantage at several points.

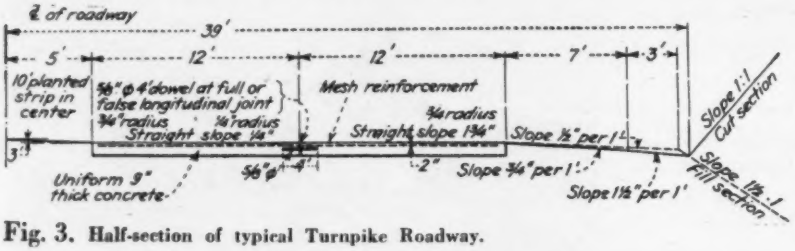


Fig. 3. Half-section of typical Turnpike Roadway.

Finishing and curing

Nearly all contractors have adopted mechanical spreaders for the dry, crushed stone aggregate concrete specified for the pavement. Several are using the Jaeger screw type spreader with which the batch, dumped in the center of the roadway, is worked in either direction by a reversible open screw. Some are using a Blaw-Knox spreader, developed on the Turnpike, which uses a transverse moving blade set at about a 45-degree angle followed by a strikeoff plate. At the edge of the slab the moving blade rotates through 90 degrees and back-tracks, the entire machine continuing to move ahead.

A 24-ft. Heltzel strikeoff was in use on one job. All machines are instantly adjustable to strike off the concrete 2 in. below surface as required for placing the reinforcing mesh.

High quality surface

Specifications permit only ¼-in. variation from a 16-ft. straight edge, permit but do not require a longitudinal float, and do not permit use of wood hand floats. All contractors used one two-screed mechanical finisher, many use two, though in some cases the lead finisher also "struck off" the top course of concrete. Most of the outfits use the mechanical longitudinal float developed by Koehring. Vibration is required only at premolded joints spaced generally at 77-ft. centers in the entire length of the roadway. Flexible shaft type easily portable vibrators are used for this service. In one area this vibration is also used along longitudinal edges of the 12-ft. lanes.

Following the machines, finishers test the slab using the edge of long redwood pieces 1 in. wide to move humps or fill holes. Transverse belting is specified but not used. Instead, general procedure is to draw wet burlap along the surface longitudinally to take out all other marks. Transverse brooming is used only in the tunnels.

Curing methods

Curing of concrete is done generally by burlap mats placed on the concrete as soon it does not mar and replaced after 24 hours with impervious Sisalkraft or similar tough fiber cemented papers laid over thoroughly wetted concrete. The paper is kept on, without wetting, for 48 hours. The paper, for a 12-ft. lane, comes in 13.5-ft. widths usually 70 ft. long and is handled by rolling on a 2 x 4 nailed to one end of the paper.

Pavers may be operated on the slab after not less than twice the time required for the concrete to attain a strength of 550 lb. per sq.in. in extreme fibers in bending as determined from test beams made and cured under road conditions. Generally the 550 lb. is attained in less than 4 days, trucks and pavers allowed on the slab in seven days.

On most contracts the rough grading was done by other than the paving contractor, the general plan being that the earth to be handled

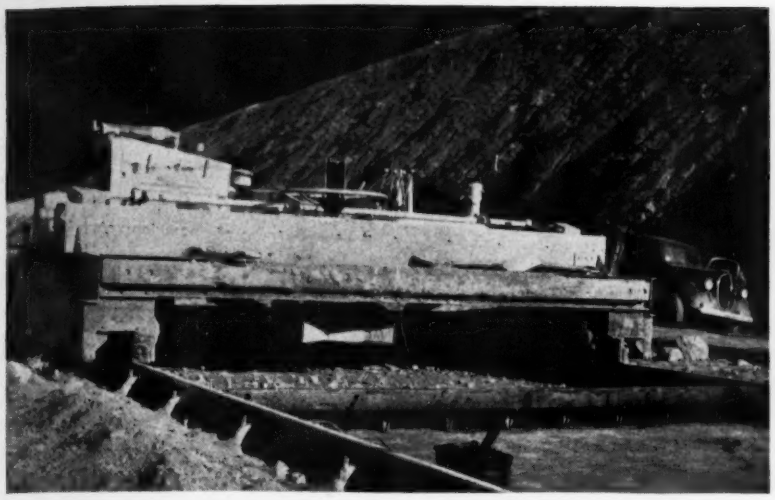


Fig. 5. New type of concrete spreader in which the blade travels transversely in front of strikeoff board, turns through 90 degrees and back tracks as machine pushes ahead.

ahead of the fine grade operation would be small. Actually, rooters, blades, even large tractor drawn scrapers are in operation to get the grade "down." A new Buckeye surface grader is at work on the western end of the Turnpike. This truck-like machine cuts the earth to about grade, throws it on a swiveled stacker belt, for truck loading or discharge on either side of the roadway.

Form graders are used by almost all contractors to cut the form line to grade. Conventional steel forms are in use, one type having two wedges at each pin to facilitate lining especially through rocky sections. A new practice, but used throughout the Turnpike, is the driving of form pins, minimum $\frac{7}{8}$-in. diam. and 24 in. long, with pneumatic hammers. Usual equipment is a paving breaker type hammer but new equipment is lighter. Portable compressors are used. Form tampers are usual equipment, only one engineer voicing any doubt as to their sufficiency. Fine grading is done by machines. Rollers compact the grade and a scratch templet is used for final check. No hauling or other activity is carried on between the forms, the paver operating just to one side of the lane being placed.

Tunnels are at present only bored for two 11.5-ft. lanes of traffic, full four-lane operation waiting until twin bore construction can be financed out of revenues. (*ENR*, June 6, p. 48). At the portals two-lane traffic east and west is forced into single lanes in each direction without a separating strip. Ten-inch concrete pavement is sloped both ways from the center and drained to the elaborate water collection system of the tunnel. A sub-base material of 9 to 12 in. of broken stone covered with about 6 in. of screenings or earth carry the slab without connection to the tunnel concrete.

Subgrade and preparation work use less machinery than is common outside the tunnels and general plan of procedure is to bring in mixed concrete by the same means and from the plants used for the lining. Finishing machinery is similar to that used outside though perhaps less of it as a hot sun does not make a critical afternoon period and no attempt has been made yet to really get production as paving is supplemented to lining work.

Labor and speed

Unskilled labor for surfacing work has come largely from traversed area of central Pennsylvania. The available local skilled help is practically all employed but contractors have not been restricted in bringing in their own organization of skilled men. Shortage of finishers was acute at the peak of the work as the 40-hour week made three shifts desirable even on 2-shift paver operations. Wages are frequently over the contract minimum of 52½c. for common, $1.00 for finishers, $1.25 for mixer operators. One contractor, John F. Bloomer of Appleton, Wis., guarantees a full 40-hour week to all skilled men and pays a bonus for all batches over 58 (66 is theoretical maximum) put through the 34E paver he uses. His yardage record is high. One paver, between two organized tunnel jobs, is using union labor, while others are operating on an open shop basis.

Close cooperation

Cooperation between engineers and contractors is generally excellent, probably because of the experienced class of men working for both the Turnpike Commission and the contractors.

Deterrents to rapid advancement have been spring rains and lack of dump trucks. Difference in equipment used makes comparisons unfair but strictly on a production per outfit basis the following contractors seem to be in the lead: Last fall Walker Bros. of Chambersburg, Pa., averaged 228 lin. ft. per 12-ft. wide 9-in. thick paving per net hour mixer operating time on a 10.3-mile four 12-ft. lane section near the eastern end of the Turnpike. John F. Bloomer of Appleton, Wis., seems to be doing best this spring having laid 2,626 ft. of 24-ft. wide slab in 21 hours and consistently laid more than 100 ft. per hour with one 34E paver. Midwest Construction and Asphalt Co. of Chicago has turned in some nice daily records using a 34E paver followed by a 27E. Almost every contractor has a claim to some particular hourly, shift, or daily record with his type of equipment.

The contract provides for one core measurement for not more than 4,000 sq.yd. of concrete and requires additional cores at 100-ft. longitudinal spacing in case cores more than ¼ in. deficient in thickness are found. Payment is not affected up to ¼ in. scant. From .26 in. to ½ in. deficiency, payments are graduated down from 90 per cent to 70 per cent of the contract price. Pavement more than ½ in. scant in depth is to be removed and replaced.

Personnel

Space limitations prevent giving individual credit to all those active in the finish rush to complete the paving before the opening date. Representative of the Turnpike hard workers are Samuel W. Marshall, chief engineer; Rodger W. Stone, chief construction engineer and John Paul, assistant construction engineer. As usual the contractors' supervising forces will get their acknowledgment of work well done by being given more work to do.

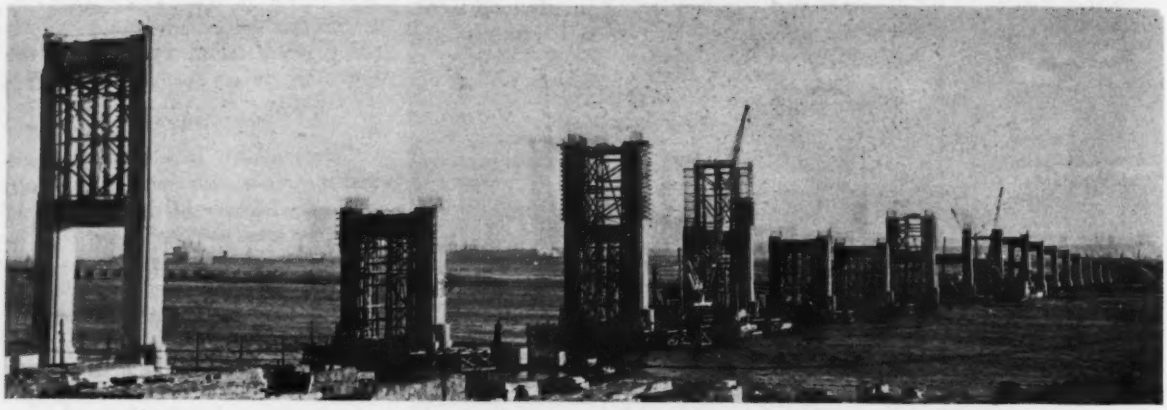


FIG. 1. HIGH PIER CONSTRUCTION ON RARITAN RIVER BRIDGE AT PERTH AMBOY, N. J.

Unusual Equipment Speeds Caissons and Piers

***Contents in Brief**—A high level crossing is nearing completion to carry the traffic of the New Jersey shore resorts across the Raritan River at Perth Amboy, N. J. Deep piers sunk as much as 85 ft. to rock as pneumatic caissons, and extending 125 ft. in the air, were constructed entirely by floating plant. The contractor assembled some unusual equipment including especially long booms for the high pier work.*

TO RELIEVE a traffic bottleneck of a low level bridge, with movable span over a busy channel, the New Jersey Highway Department is completing construction of a 52-ft.-wide roadway 4,386-ft.-long bridge over the Raritan River at Perth Amboy. The river at this point is nearly 2,000 ft. wide and is shallow, having only a narrow channel suitable for navigation but requiring a 200-ft. channel opening with overhead clearance of 135 ft. above high tide.

Construction of the bridge is divided into six principal contracts, two of which, the river pier caissons and river pier shafts reaching from 85 ft. below tide level to 125 ft. above, form a considerable part of the total construction. The Peter F. Connolly Co. has successfully completed both contracts.

In general, water at the site is about 6 ft. at low tide with mud to an average depth of 50 ft. except near the banks. Underlying the mud is a layer of fine sand followed by decomposed trap rock changing in a few feet to hard trap suitable for any load. Design contemplated founding pneumatic caissons on the softer rock and cutting out to solid rock to provide the foundation necessary for the high, narrow piers. Shore piers are founded on spread footings in clay and hardpan material except that the north riverside pier rests on timber piling to hold the thrust of the approach fill.

Design of the caisson was left to the contractor, subject to the engineer's approval, except for outside dimensions. Fig. 2 shows plans for the working chamber. These chambers generally exceeded by 6 in. the width and breadth required by the design to permit some tolerance in sinking. Working chambers for each of the 10 piers were shop-fabricated in as large sections as could be shipped by rail, then assembled on a dock with all seams welded watertight and set into the water by a floating derrick for towing to the bridge site. Maximum size chambers were 22 ft. 6 in. by 61 ft. 6 in. by 9 ft. 6 in. deep and weighed 40 tons. About four feet of freeboard was available when afloat.

Water at the bridge site was too shallow to permit water work so a suction dredge was employed to make a channel 150 ft. wide from the upstream line of the piers for the full length of the bridge. This left 100 ft. on one side for maneuvering caissons and equipment to each pier site. Fifty-foot-long timber piling was driven about 30 ft. below the dredged channel to form a work and guide platform at each pier, into which the working chamber was floated. There, the structure was built up by addition of vertical 6-in. 12-lb. I-beams at 4 ft. c.c. and covered with 3x12 plank, bolted to the uprights and caulked watertight. The vertical steel posts were supported against inward water pressure or outward concrete pressure, as the case might be, by rings of 10x12 timbers at 5 ft. c.c. as shown in Fig. 4. Three 9-ft.-diameter dredging wells are formed in each caisson by steel pipe. The wood jacket seen in the photo forms a key for holding pneumatic shaft equipment when required and subsequently for sealing the wells. Thirty-two 2-in.-diameter pipes are installed in the exterior walls of the structure for use as jets to loosen material and assist sinking. Interior frames were removed as concreting progressed, the vertical posts and sheathing of course remaining in place.

Construction procedure was to dig through the open dredging wells with clamshell buckets on floating derricks as long as the box settled uniformly. Concrete was placed as the weight was

required. When the caisson reached material that could not be penetrated by digging and jets the wells were decked over, usually with a poured concrete platform, and a 3-ft. shaft for a man lock and two 4-ft. shafts for muck locks were set. Air pressure was then put on, never exceeding 34 lb. for the maximum 83-ft. depth, and excavation proceeded to the required quality of foundation. All piers were kept well within location tolerance limits. The deepest pier, El.-85.5, "sealed" sufficiently in the penetrated material to permit completing as an open caisson. Some blasting was required to prepare a satisfactory surface on the hard bottom rock in all caissons.

River pier shafts

The same contractor was successful bidder on the shafts for the river piers so much of the work could be performed with the same equipment. General plan was to handle all forms and reinforcing steel into position with floating derricks but place only the pier top concrete with the derricks. An ingenious arrangement was used to assemble the complete cage of reinforcing for the high (maximum 46 ft.) lift of concrete in the columns, no construction joints permitted. Secondhand I-beams were ripped to make the angle frame shown in Fig. 3. The complete column cage consisting of 1¼-in.-square rods, some as much as 58 ft. long, was assembled on the deck of a barge with hooked bolts holding the bars to four angle frames. The cage, largest one about 10x12 ft. and weighing 7 tons, was then picked up by derrick and set in place giving a framework to proceed with the forming.

Similar piers, one reuse on the same pier, working high in the air, and heavy equipment available for handling were factors which indicated the desirability of panel forms. Briefly they were ¾-in. plywood faced, 2x6 vertical studs at 10½-in. centers, with 3x6 wales at less than 3 ft. c.c. Ties were 9,000 lb. working load at about 3-ft. centers each way in each wale. Fig. 3 shows a typical condition with the inset corner panel used to achieve the architectural effect desired and make the "standard" panels fit varying size piers.

Feature of the contractor's plant was a combined compressor, hospital and derrick boat. Mounted on an 87x30x8-ft. steel hull with outside spud wells this rig, spudded into the mud, held one side of the caisson into the guide frame until it was sunk sufficiently to prevent movement, supplied air for expelling water from the caisson and did its own excavating. (The derrick was not available for early use as it was mounted principally for the high shaft construction of the second contract). The derrick, of the full whirler type, powered by a 12 in. x 14 in. cylinder engine using air from the compressors, is

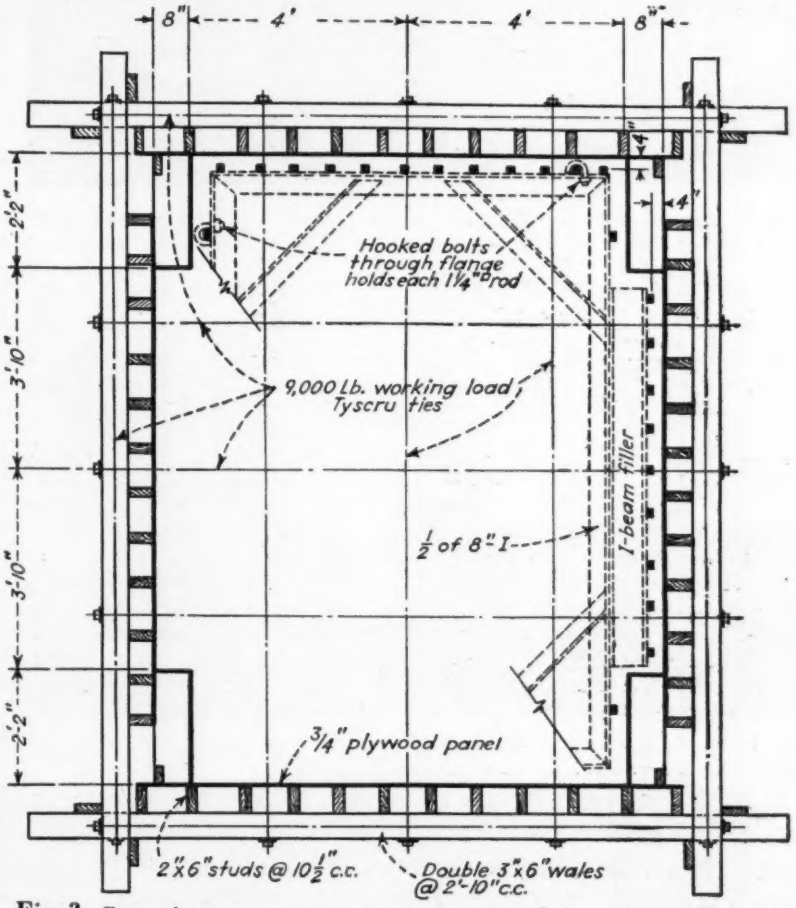


Fig. 3. Forms for upper section of columns. Dotted lines show welded steel frame used to space steel and permit handling into place as a frame.

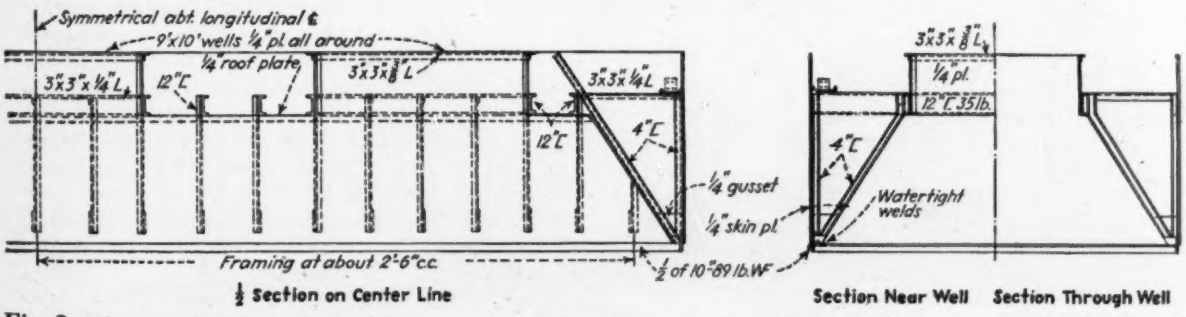


Fig. 2. Working chamber as designed by contractor. Steel sections were assembled on dock, set in water by derrick and floated to site with about 4-ft. freeboard.

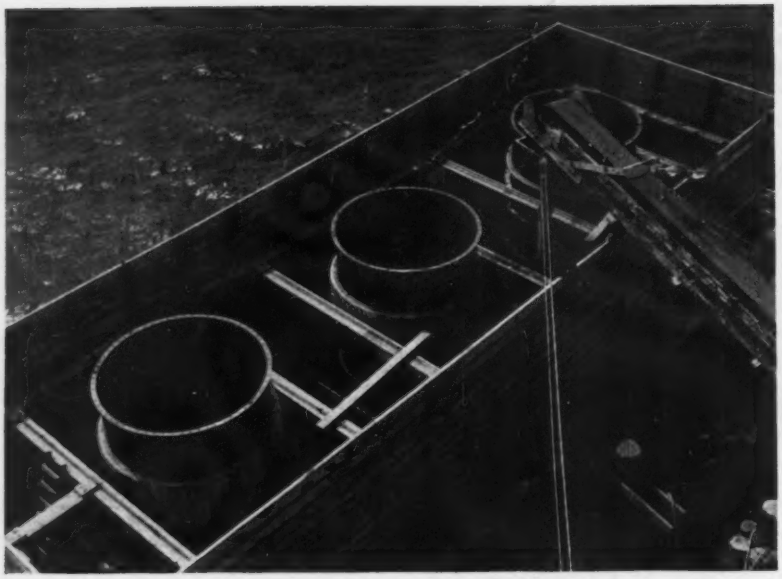


Fig. 4. Caisson ready for additional concrete. Walls are 3x12's caulked tight. Wood interior frames are at 5-ft. centers and are removed as concreting progresses.

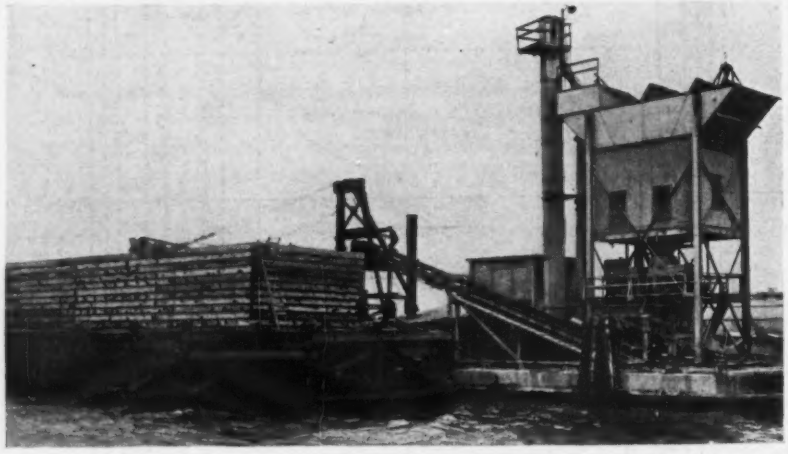


Fig. 5. The Peter F. Connolly Co. mixer plant. Note low center of gravity. Aggregates are fed to the bin by a separate floating derrick.

Fig. 6. Concreting the high piers. 90-ft. boom on the derrick has been extended to 150 ft. by a steel extension 100 ft. long slipped down over the fir boom.

mounted on a gantry with boom pins 20 ft. above the water. Its 135-ft. boom gave the rig a reach of more than 150 ft. in the air. Two diesel-driven low-pressure compressors supplied 3,300 cu.ft. of air per min. for the caissons and another diesel produced 750 cu.ft. at 100 lb. for operation of pneumatic equipment. A doctor's office and hospital lock occupied one end of the boat. Compressors and the doctor were housed in an insulated steel building.

A relatively small barge carried the concrete plant. Two one-cylinder non-tilting mixers were mounted under a four-compartment (cement, sand and two sizes of gravel) weigh batcher bin. A separate derrick boat fed aggregates to the plant from barges. Cement was shipped by rail in L.C.L. bottom-dump containers to a water-side track, where it was dumped by derrick into a 400-bbl. tank on the mixer boat. Screw conveyor and bucket elevator transferred it from the tank to the bins over the batcher as required.

Delivering the mix

The mixers discharged into a hopper feeding onto a 24-in. belt raising the concrete about 15 ft. in its 40-ft. length and feeding onto another belt, swivel-supported from an A-frame which permitted discharge at any point in a 180-deg. swing and 12 ft. up or down. For the pier shaft contract an 87-ft. builder's type tower hoist was mounted to take the concrete from the mixers to any desired height where it dumped on one of the 40-ft. conveyors which has one end mounted on the tower, the other supported on a boom frame swiveled from the tower. This rigging permitted placing two-thirds of the shaft concrete; the rest was placed by a one-cylinder bucket on a crane. As usual, booms on the equipment were not long enough but that did not stop the contractor. Starting with a 90-ft. 24-in.-diameter Oregon fir stick on a steam powered lighter he added a 100-ft. steel mast 42 in. square at the center which slid down 20 ft. over the wood boom giving a usable length of 150 ft. above the deck. Fig. 6 shows the rig in action placing concrete. Another boom was extended in a similar manner.

Construction cost

Construction of the bridge is expected to cost about 4½ million dol-

lars of which 45 per cent is a PWA grant. Work is divided into six principal contracts undertaken by the following concerns:

1. Caissons, 10 River piers—P. F. Connolly Co.
2. Shafts, 10 river piers—P. F. Connolly Co.
3. South approach, 8 piers—Felhaber Pile Co.
4. North approach, 12 piers—J. F. Chapman & Son, Inc.
5. Steel superstructure—Bethlehem Steel Co.
6. Roadway deck and lighting—John G. English, Inc., and Joseph Nesto Co.

Personnel

For the river pier work described above H. J. Deutschbein Co. was associated with The Peter F. Connolly Co., with Peter F. Connolly in direct charge of the Work. George V. Salle and Ralph R. Buscell were superintendents and A. Byron Hunicke engineer at the site.

Representing the New Jersey State Highway Department E. Donald Sterner is commissioner, James Logan, highway engineer, Morris Goodkind bridge engineer and C. L. Appleton resident engineer on the bridge.

Contractor-Engineer Relations of a Lifetime

F. L. COPELAND

Vice President, Bates & Rogers Construction Corp., Chicago

Contents in Brief—After a third of a century with one of the leading and most respected construction firms of the country, Mr. Copeland writes of the engineers he has encountered. He recalls the early days of the century when the superintendent and engineer for a job were isolated from their colleagues and worked out between them the answers to all field problems. Perhaps distance lends enchantment to the view, but the author prefers the tough going of the earlier days to the red tape and multiplicity of engineering "bosses," regulations, and financing in which our present public contracting is entangled.

FROM 1905 TO 1940 thirty-five years of general contracting with one concern on work from Maine to California have slipped by all too quickly. I have been asked after all these years of doing work for railroads, the federal government, state highway departments, cities, counties, utility corporations and private companies to jot down a few observations as to the relations we have had with the engineers who have been our "bosses."

In the heat of youth on both sides scraps (none very serious) were numerous and often intense, but as we grew older a more logical attitude governed and we learned to see better the other fellow's viewpoint. In all my varied experience, as I look back over it, there were but a very few engineers who, in my opinion, did not honestly represent the owner. The great majority felt they were being paid to look after their employer's business just as I felt I was being paid to look after my company's business.

It has often been observed that the friendships formed in the old days between engineers and contractors, born of close association, greatly outnumbered and outlasted those between the contractors themselves. I always tried to keep my opinions of engineers impersonal on the work; I fought hard for what I believed right, but those of us who wished to be fair gave the engineer credit for sticking to what he believed the correct decision.

Fig. 1. Engineers led a rigorous life on the Milwaukee R. R. in Idaho, 1908, and they were of necessity independent.

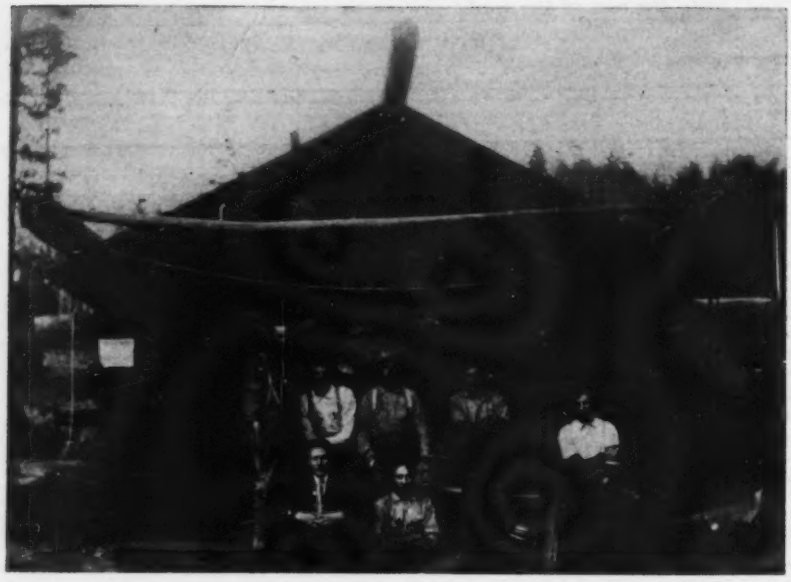


Fig. 2. Engineers and contractors lived together at Pee Dee, Idaho, camp on the Milwaukee R. R. in 1908. The spruced-up gent seated at the left—note the stiff collar and tie—is the author.

From 1905 to 1917 I worked on the railroads of the Middle West and Far West, on bridges, grading, tunnels, terminal construction and tracks, first as timekeeper, then civil engineer, foreman, assistant superintendent, superintendent and general superintendent, from Illinois to the Pacific Northwest, and on dams in Wisconsin and Idaho.

First job off center

My first job was at Danville, Ill., on a new railroad built in 1905. I can well remember the pair of abutments built by using the center of the northbound track as the centerline of structure instead of the centerline between tracks, and the consternation of all of us when the fill approached the bridge and it became evident something was radically wrong. In this earlier day grading was largely done by station men and concrete was mixed by hand on a platform, the materials were secured as best they could be, and the water-cement ratio was maintained by a laborer with a barrel and a pail. Generally pit-run gravel was obtained from a nearby bank; if it happened to be dirty or poorly proportioned, more precious cement was added.

Engineers of this earlier day are now classed as "old-timers." They did the best they could to get a good job with the materials available. The inspector on the job—it had to be a real big job to have more than one—was active to o.k. the foundation, rough-check the forms, and he generally helped spade the concrete or otherwise rendered some service to push the work along. The stove in the little job shack, which we called an office, with the coffee pot shared by all, was an institution.

Engineers on their own

The old-time engineer was often a long way from communication, miles ahead of the end of track. His chief's office was weeks away by mail, there were no phones and no telegraph, so there was no one to help him make decisions except a bunch of younger engineers and us contractors. If the plans didn't fit the job he had to act, and act he did, quickly and surely. A fine crowd of men these pioneer engineers were. I always think of John F. Stevens as the dean of them all. The engineers he trained include many of our leaders today —railroad presidents, chief engineers, etc.—resourceful, of sound judgment, honest and fair. He is a symbol of the "old-timer." There are many who had his ideals, his character, and a few his ability.

They respected the contractors too. I remember when in 1909 the engineers with whom we worked elected the then president of our company, Onward Bates, to the presidency of the American Society of Civil Engineers. A contractor picked by his bosses for the highest engineering honor!

The engineers were strict and often got real tough. They demanded the best we could produce; they seemed at times merciless with the slacker or the crook; but they were square and they knew their business thoroughly. They very seldom expected us to do things for them outside of our contract without adequate compensation. They wanted the contractor who did a good job to make a fair profit. These old-time engineers and the contractors built the railroads of the western country, some of its best and biggest bridges, dams and tunnels, and today can still be proud of their work. They had no modern machinery, no high-powered designs, but as I travel over the country and look at their work I still marvel at their ingenuity, their common-sense engineering and their construction knowledge.

In 1917 the country went to war and the Government called us. In '17 and '18 we built Camp Grant at Rockford, Ill., a storage depot at New Cumberland, Pa., a nitrate plant at Toledo, Ohio, all for the Construc-

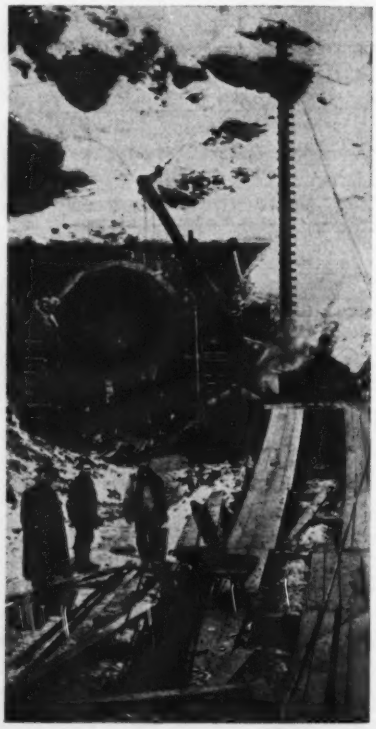


Fig. 3. On the Northern Pacific in Idaho, 1906. The steam-driven concrete mixer was charged by weelbarrow batches.

tion Division of the Army. We also built cribs, dikes and 85 miles of railroad at the Hog Island shipyards. The Construction Division personnel were practical civilian engineers and architects and some regular Army men who knew their work. They worked, as we did, day and night, through Sundays, regardless of weather, to provide camps and other necessary structures. Many of these engineers were men who had left big jobs to serve their country.

Following the war we entered the new field of concrete highway construction and there had our first experience with the grafting engineer. Much of this has been cleared up now, but it was a distinct shock to meet an engineer who was not absolutely honest. After a short venture in the unfamiliar highway field we returned to railroad and public utility construction. Then came the financial crash of '29.

Sudden change

The transition from private work for old friends to the new order of things was rapid and severe. The reputation of our company and the fact that over a long period of years we had made many steady customers and friends were worth nothing almost overnight. The good and the bad contractor were now on an equal footing, and always the low bidder who could get a bond built the job. Today specifications state that the successful bidder must: (1) be experienced on the class of work to be constructed; (2) have adequate plant and organization; and (3) satisfy the owner as to his financial ability to carry the work through to completion. Yet job after job is let to people who ask us for an organization, who want to rent our plant as they never had any, and who never had any experience whatever on the kind of work on which they bid.

Under this regime we did considerable state highway work financed by federal aid, some government work, several dams, a large bridge or two, and some tunnel work for different states. In general we have received fair treatment from the top engineers; but the inspection usually has not been very practical. There are all too many agencies involved, each trying to dictate the management of the work. We recently finished a job in the East let by a state highway department which contributed nothing to the cost of the work. The chief engineers of three different railroads, the PWA, Bureau of Public Roads, the city engineer, and the state highway engineers all had a hand in it and we couldn't possibly please them all because they demanded different things. On this job the specifications called for heating poured concrete until April 15. On April 12 the heat record was broken with a temperature of 82 deg. in the shade, yet one of the engineers representing one of the interested parties demanded we steam-heat the concrete that day.

Sprinkling concrete in rain

Another similar case (most amusing until it's your money involved) was the lad who had us sprinkling water to cure concrete on a dam during a heavy rain. When we asked why God wasn't doing a good job of it, he replied: "The specs say the contractor shall do it, nothing is said about God." He contended he was well within his rights to make us continue sprinkling in the rain.

With a multitude of agencies trying to run a job it becomes a real task to make progress. The project engineer is usually a good man, but due to the number of engineers now on a job it is inevitable that some are not competent. One of the worst of these is the man—often a young inspector just out of college—who reads only "as directed by the engineer" in the specifications and proceeds to direct the job not only as to results but also as to methods. The top engineer is often further harassed by having his estimates for something which is fair held up while someone contributing to the cost of the work seeks to find a legal loophole to avoid payment. Too often the man who should be on construction is spending his time making "red tape" reports while inexperienced men direct the contractor.

In these days of budgets, money for necessary extras is often not available, and the engineer is required to chisel his way through to complete a job. An unfair procedure is failure to give adequate time allowances for changes of plans and then to extract a penalty. In one case we paid 17 days' damages two years ago on a project not yet in use, as right-of-way essential to operation has not been bought. A most objectionable engineer is one who requires better than specification materials and workmanship then defends his stand on the ground that it is his duty to get the best possible job. Worst of all is the owner or engineer who falls down on his part of the contract and by technicalities avoids paying the contractor for delays and damages. Such a case is illustrated by a dam we built, where the owner agreed to move a railroad from the site by a certain date. We finished half the dam, then lost a whole year waiting for the owner to move the railroad out of the way. He failed to fulfill his part of the agreement, and yet refused to pay for the delay.

Fig. 4. Hammering them down with a steam hoist and drop hammer, Milwaukee R. R., Idaho, 1908.

Contractors also to blame

Some of the contractors' woes are entirely their own fault. A lot of us became accustomed to doing work for engineers we knew well and knew exactly what they expected and wanted, so we paid little or no attention to clauses in the specifications which we knew would never be enforced. A reclamation engineer once said to me "Most of our troubles with contractors start when they read our specifications and then refuse to believe we mean what we say. If the contractor would get it into his head that we must enforce to the letter all clauses in our specifications, and price his work accordingly, better

prices would prevail and wrangling would cease." Of course he was right.

Another thing we old-time contractors face is the change in contracting procedure. Our old clients picked their bidders, and we had to be honest or we were kicked out. We were treated with respect and built a good job or we lost our customer. Only such firms survived as delivered the goods. Today the engineers haven't the slightest authority to pick their contractor. If a crooked, incompetent, inexperienced contractor is low bidder he usually gets the job and proceeds to pull the job down to his level if he can.

The contracting business is as ethical as any other business. The average contractor, though some engineers still regard him as a moral and social inferior, is simply a plain, generally educated and refined business man trying to make a living and make his business pay. He is as much entitled to a profit as a banker or a grocer. In general most engineers treat him as such, but there are too many younger engineers coming along who have every contractor labeled as crooked and incompetent. I don't believe these engineers will last—if they do, we won't.

I often think how glad I am that I've enjoyed the friendship, and I'm sure the respect, of so many engineers. They are a fine lot of men, but I'm glad I am nearly through instead of just starting out.

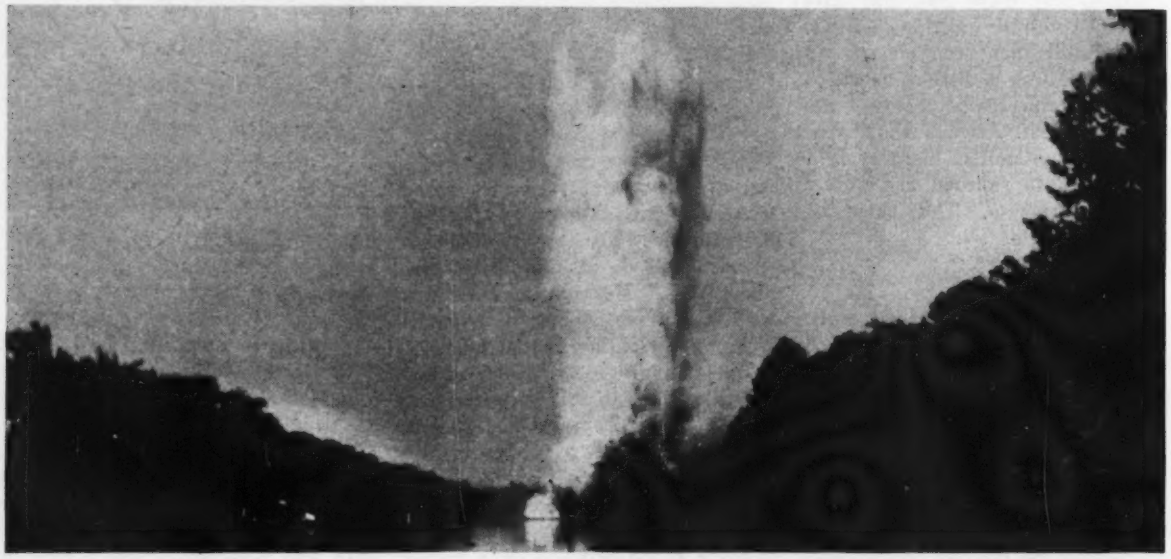


FIG. I. DYNAMITE BLASTS A TRENCH FOR AN INVERTED SIPHON ACROSS THE LEHIGH RIVER.

Mountain Water For Bethlehem

Contents in Brief—To replace a supply now obtained from deep wells and the Lehigh River, $3,900,000 is being spent by Bethlehem, Pa. to develop a mountain supply 25 miles from the city. This will not only provide good water but will permit an annual saving of about $55,000 in pumping costs.

GOING TWENTY-FIVE MILES northwest of its city limits, Bethlehem, Pa. (population 70,000), is developing Wild Creek as a gravity water supply to replace a supply now obtained from deep wells and the polluted Lehigh River. Major items of construction include: a rock and earthfill dam 140 ft. high; 2 miles of tunnel; a 23-mile concrete pipe line; 2 miles of smaller pipelines and three structures for control and operating purposes. The project was started in January 1939 and is expected to be completed by November at a construction cost of approximately $3,900,000.

Of the present daily consumption of nearly 7 mgd, the deep wells furnish 800,000 gpd, about half what they did when they were developed in 1908. This reduction, plus the fact that the Lehigh River is badly polluted by industrial wastes, created the need for a better supply. Furthermore, water from both sources is quite hard, that from the wells averaging 220 ppm and that from the river 179 ppm. Because of these conditions an improvement of the city's supply has been under consideration since 1920.

Five improvements studied

Five possible improvements were studied: (1) remodeling the present filtration plant and constructing additional filters; (2) developing an additional supply from underground

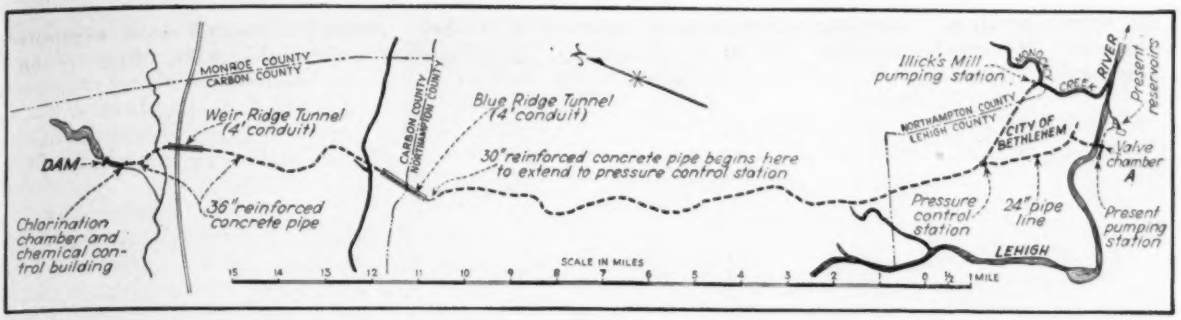

MONROE COUNTY
CARBON COUNTY
Weir Ridge Tunnel (4' conduit)
DAM
Chlorination chamber and chemical control building
36" reinforced concrete pipe
CARBON COUNTY
NORTHAMPTON COUNTY
Blue Ridge Tunnel (4' conduit)
30" reinforced concrete pipe begins here to extend to pressure control station
Illick's Mill pumping station
LEHIGH RIVER
Present reservoirs
NORTHAMPTON COUNTY
LEHIGH COUNTY
CITY OF BETHLEHEM
Valve chamber A
Pressure control station
24" pipe line
Present pumping station
LEHIGH
SCALE IN MILES
15 14 13 12 11 10 9 8 7 6 5 4 3 2 1 0 ½ 1 MILE

Fig. 2. From a 300-acre reservoir of four billion gallons capacity formed by an earth fill dam 140-ft. high, water flows through two 38-in. steel pipes to chlorination chamber and chemical control building. After treatment water travels through 23 miles of pipeline and two miles of tunnel to a control station at the city's edge.

sources; (3) using the Delaware River; (4) tapping the tributaries of the Delaware, and (5) obtaining water from Wild Creek, a tributary of the Lehigh River. With the quality of the Wild Creek water such that only chlorination and treatment with lime will be required, and with this gravity supply making possible an annual pumping saving of $55,000, the Wild Creek development was decided upon when a PWA grant supplying 45 per cent of the total cost became available. This water has a hardness of 7.9 ppm. An additional advantage is that nearby Tunkhannock Creek can be developed in the future without great cost to provide a total of 35 mgd.

Two-year supply storage

When the storage dam is completed, a 300-acre reservoir of four billion gallons capacity will result to provide sufficient water for nearly a two-year supply at the present rate of consumption.

The dam will be 140 ft. high with a crest length of 1,300 ft. and a crest width of 30 ft. The thickness of the base from downstream to upstream toe will be 1,100 ft. At the west end of the dam a concrete spillway, designed for a maximum discharge of 17,500 cfs, is being built. The top of the spillway crest is at El. 820 with the crest of the reinforced concrete wall forming the downstream end of the stilling pool at El. 682. The design of the spillway was adopted following model studies made at Lehigh University under the direction of A. I. Ippen, assistant professor of civil engineering.

The intake tower is being constructed through rock at the east end of the dam and water from this point for the first 1,600 ft. will flow through two 38-in. steel pipes in tunnel and open cut to the chlorination chamber and chemical control building. The first 980 ft. of this pipe line is built within the 12-ft. concrete lined tunnel extending through the hill forming the east abutment of the dam; the final 620 ft. is built in an open cut.

After treatment with lime and chlorine the water will flow, as shown by Fig. 2, through a 36-in. reinforced concrete pressure pipe to the portal of the Weir Ridge tunnel, which is 3,660 ft. long. This tunnel, built through solid rock, is lined with concrete and will provide a 4-ft. conduit. From here the water flows through a 36-in. concrete pipe to the Blue Ridge tunnel which is equal in size to the first and 5,230 ft. in length. From the Blue Ridge tunnel a 30-in. reinforced concrete pipe transports the water to the control station where the flow will be divided, a portion flowing through a 20-in. steel pipe to the Illick's Mill pumping station at the northern section of the city and the remainder through a 24-in. line under the Lehigh

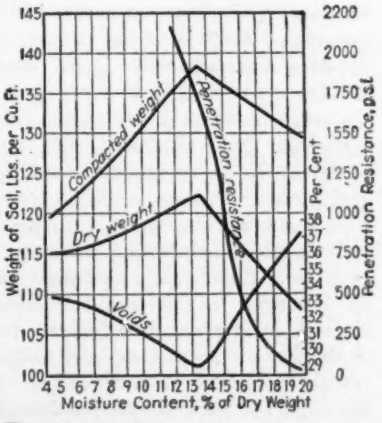


Fig. 3. Typical analysis of fill material. Penetration resistance measured with plasticity needle after 6-in. layer had been compacted by dropping 5½-lb. hammer 25 times from 12-in. height.

River to the South Side Reservoir.

Grouting under core wall

Construction of the dam is well advanced as shown by Fig. 4. Also shown is the concrete core wall 10 ft. high built in a trench cut 3 ft. into the sandstone foundation, as is the concrete spillway. Along the center line of this core wall, grout holes averaging 30 ft. in depth are provided every 3 ft. and grouting is done at pressures of 50 to 75 lb. per sq.in. Steel pipe extending upward through the wall allows this work to be done after the wall has been built. During construction the flow from Wild Creek is carried under the dam by a 14-ft. reinforced concrete conduit. A 36-in. cast iron pipe will go inside this conduit later to provide drainage of the reservoir below the bottom intake level.

The earthfill portion of the dam is placed in 6-in. layers with a moisture content of between 10 and 14 per cent by dry weight, each layer rolled eight times with sheepsfoot rollers which produce a pressure of 1,500 lb. per sq.in. As shown by Fig. 3, this results in a density of 130 to 140 lb. per cu.ft. Data shown in Fig. 3 was obtained from tests performed on the average material available near the site.

Use of the toluene method to determine the moisture content of the soil has been a big aid in obtaining maximum soil density, since this method gives the moisture content much quicker than ordinary methods.

Tunnel construction has been simple, being mostly through firm rock. The 3,660-ft. Weir Ridge tunnel, which was holed through on Nov. 20, is now almost completely lined with concrete. The 5,230 ft. Blue Ridge tunnel was holed through April 16, and lining work has begun. Con-

Fig. 4. Earthfill dam is being constructed in 6-in. layers. It will be 140-ft. high with a crest length of 1,300 ft. and crest width of 30 ft. when complete. During construction Wild Creek flow carried under dam by reinforced concrete conduit.

struction of the 23 miles of reinforced-concrete pipe has also been completed, this work having been done by the open cut method.

Siphon construction unusual

Placing the 24-in. cast steel inverted siphon across the Lehigh River called for some unusual construction because the contractor, instead of building a cofferdam to do the work in the dry, chose to place the pipe under water. First a trench was prepared by placing 60 per cent dynamite directly on rock rather than in drill holes; this was done because it was found the water and mud served as a very good cap. A 160-ft. column of water resulting from a 600-lb. charge used in such a way is shown by Fig. 1.

After the trench was excavated with a clamshell bucket, lengths of pipe were lowered from a barge and placed by divers. E. R. Morehouse & Co. of Camden, N. J., performed this undertaking under subcontract to the Lock Joint Pipe Co.

The project is under the direction of the Bethlehem Municipal Water Authority, bonds are to be liquidated in 25 years. For the Authority, R. L. Fox is chief engineer; L. M. Fraivillig, principal assistant engineer; J. G. Conrath, assistant engineer; and J. S. Richards, H. J. Evans, and F. Narzisi, resident engineers for the dam, tunnels, and pipeline respectively. The late Thaddeus Merriman was an advisory consultant. G. Edwin Pidcock is chief resident inspector for PWA.

C. W. Good Construction Co., Lancaster, Pa., has the contract for the dam and is represented on the job by C. W. Good. The Lock Joint Pipe Co., holding the contract for 23 mi. of the pipeline, is represented by J. A. Tapley. Harry DePue represents DePue & Beers, contractors for the clearing and grubbing work. Kaufman Construction Co., Philadelphia, the tunnel contractor, is represented by Frank Depman. Malone & Sons, Lancaster, Pa., contractors installing 2½ mi. of 28-in. steel pipeline, are represented by James and John Malone.

Advantages of Box Abutments

The trend toward increased length of roadway spans of grade-separation bridges has led to extensive use of the box type of abutment, which includes the abutment, curb pier, footings and sidewalk span as a monolithic structure. This was noted in the masonry report at the recent meeting of the American Railway Engineering Association, which pointed out that in designing such abutments on the rigid-frame principle it is necessary to give careful attention to details. Where separate or independent piers are placed at the curb lines to support the roadway and sidewalk spans, difficulty has been experienced in design owing to insufficient bearing areas for the steel shoes on the piers, as well as interference between the footings of slabs of piers and abutments. In addition, the deflections of the two spans produce unsatisfactory joints over the piers, particularly in skew spans. To overcome these objectionable features the box abutment was developed. In its design, the location and form of construction and expansion joints are important, in regard to both stress distribution and waterproofing. Reinforcing steel must be so arranged as to avoid sudden changes in stress and to insure adequate facility in placing and tamping the concrete. Waterproofing of the sidewalk span should be treated as on a bridge deck, with non-corrosive metal waterstops at all construction and expansion joints; and provision must be made for expansion movements.

The only disadvantage in this type of abutment appears to be in preventing future changes in the curb lines. On the other hand, advantages are listed as follows: (1) excellent distribution of loads on foundations; (2) adequate bearing area for spans; (3) ease of providing for contraction and expansion; (4) avoidance of a short steel sidewalk span of uneconomical proportions; (5) improved waterproofing conditions; (6) longitudinal rigidity to resist braking and other traction loads; (7) economy in construction; (8) elimination of structural difficulties in designing joints between adjacent spans, and also distortion of floor system members of skew spans continuous over the curb piers. As to appearance, the box abutment is said to lend itself to architectural treatment and may produce a pleasing structural balance for grade-separation structures having spans of great or moderate length.

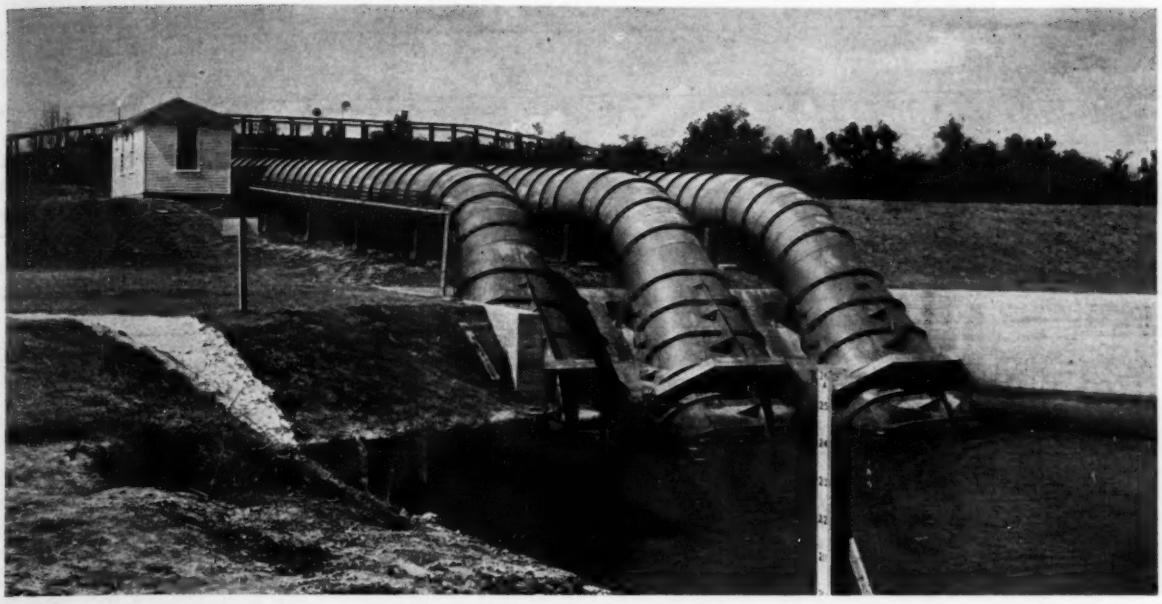


FIG. 1. BATTERY OF 9-FT.-DIAMETER SIPHONS ON ST. FRANCIS RIVER LEVEE IN ARKANSAS.

Lifting a River Over a Levee

MAJOR JAMES D. ANDREWS, JR.

Assistant to the District Engineer, Memphis Engineer District, West Memphis, Ark.

Contents in Brief—Departing from conventional practice in transferring water over a levee, the Army Engineers have installed a siphon to replace a sluiceway on the St. Francis River project. Extensive research on siphon hydraulics produced new findings which resulted in a structure with a discharge coefficient of 0.97 as contrasted to the previously established value of 0.75. Hydraulic and structural design as well as performance data are given in this article.

A SIPHON INSTALLATION recently completed at Marked Tree, Ark., to lift a river over a levee, represents an engineering design which is unique in application, and which incorporates new information on the hydraulics of siphon action. The installation, consisting of three 9-ft.-diameter steel siphons capable of discharging 2,600 cfs, was designed by the U. S. Army Engineers as part of the work on the St. Francis River flood control project. Tests indicate that a discharge coefficient of 0.97 can be achieved, which is much higher than that obtained on large siphons previously built.

The Marked Tree siphon is located about 40 miles northwest of Memphis, Tenn., at the foot of St. Francis Lake (Fig. 2). During flood periods a considerable amount of water left the main channel of the St. Francis near the location of the present siphon and flowed through a low area called Sand Slough and then back into the river near Wittsburg. To alleviate flood conditions, local interests formulated a plan to dam the river just north of Marked Tree and force all the water through Sand Slough. Since the river is navigable and carries considerable traffic, the War Department granted a permit to divert the flood waters (flow above El. 210) on the condition that a lock and a sluiceway should be built at the point of diversion, the sluiceway to carry the normal flow of the river through the levee and into the original channel.

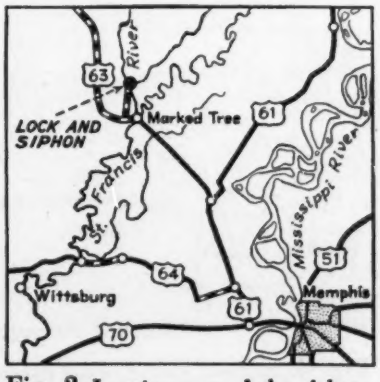
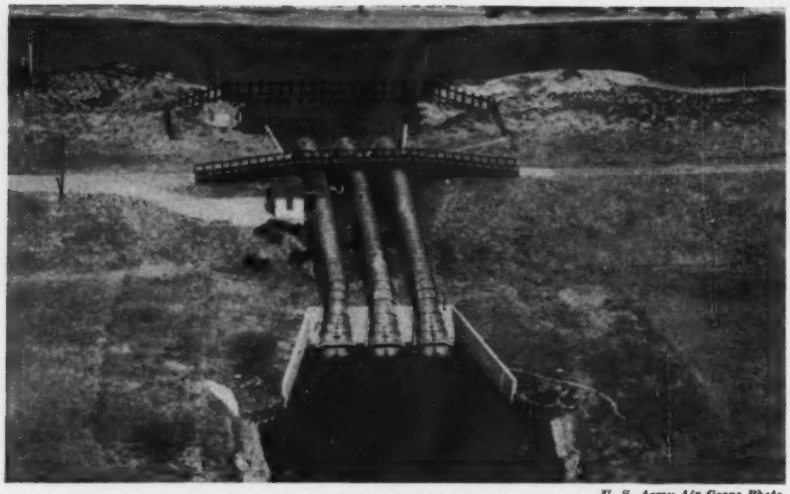


Fig. 2. Location map of the siphon.

To comply with these terms the entrance to the overbanked floodway was dammed to El. 210, making the floodway operate automatically for flows above this level. Since the old channel is filled with a flow of 2,600 sec.-ft. the permit stipulated that the sluiceway should be of such capacity that it could discharge the flow when it was available. In 1926, the floodway, lock and sluiceway were placed in operation.

In 1933 the foundation of the cul-



U. S. Army Air Corps Photo

Fig. 3. Water is taken from the river (in the background) and lifted over the levee by means of siphon action.

vert was undercut but it was continued in operation until 1936, when continued erosion and resultant caving at the outlet seriously endangered the safety of the structure. In 1938, while repairs were being made, the levee adjoining the sluiceway crevassed and the sluiceway itself settled vertically. When plans were studied for its replacement it was determined that a siphon or a battery of siphons had distinct advantages over the culvert type of sluiceway, this decision marking a radical departure over former methods of transferring water across a levee. The structure was designed by the hydraulic section of the Memphis Engineer District under the supervision of the writer. Construction of the siphon was under the immediate direction of the district engineer, and the supervision of the president, Mississippi River Commission, Vicksburg, Miss., and the Chief of Engineers.

The siphon possessed several advantages over a sluiceway because of the presence of foundation material which tended to become "quick" when saturated; a siphon structure would be less affected by adverse foundation conditions, it would have a greater factor of safety under identical conditions and it cost about $72,000 less. These two conditions—a greater factor of safety and lower cost—were the main considerations in selecting the siphon.

The principal parts of the installation included: excavated inlet and outlet channels, reinforced concrete inlet and outlet basins, three 9-ft.-diameter steel siphon tubes, water and vacuum pumps with piping, valves and motors, a reinforced concrete footing for the siphon tubes and a small reinforced concrete cut-off wall (Fig. 4).

Construction of the inlet and outlet channels, all concrete and earthwork and installation of pumps were done by the List & Weatherly Construction Co., of Kansas City, Mo. Fabrication and installation of the steel siphons were by hired labor.

Structural design features

The 130-ft.-long inlet channel has a 64-ft. bottom width, and 1 on 2 side slopes; the last 30 ft. adjacent to the inlet basin are riprapped to prevent any erosion. The inlet basin, of reinforced concrete, is 45 ft. long and 68 ft. wide at the upstream end and 60 ft. wide where the siphon tubes are supported. A cellular steel sheetpiling was incorporated in the design of the basin to confine and thus stabilize the foundation. In addition, the basin was so designed that, should the foundation settle, it could span the distance between the walls of the piling without failure. Since the basin is submerged at all times it was not necessary to consider uplift pressure in design.

The possibility of vortex action within the basin was a source of concern and necessitated considerable original research to produce a streamlined flow. The problem was to produce a flow without obstruction or interference of one suction with the other. Such interference would give rise to eddy action, vortex action and the formation of suckers which would permit the rhythmic gulping of air through the siphon and would create difficult vibration problems throughout the structure. To overcome this possibility, two factors had to be satisfied; namely, (1) placing the tubes to prevent interference; and (2) providing sufficient depth of basin to keep approach velocities low over the basin.

The spacing was fixed at a distance considered correct, and the basin floor was set at an elevation 5 ft. below the entrance to the tubes. This depth, when based on the design flow of 2,600 cfs, gave an approach velocity of 5 cfs at the entrance, which was not considered excessive and at the same time gave a water seal of greater depth than the approach velocity head for the entire range of siphon operation. To date there has been no evidence of vortex action with one, two or three siphons in operation, which leads the writer to believe that this concept of design is equally applicable

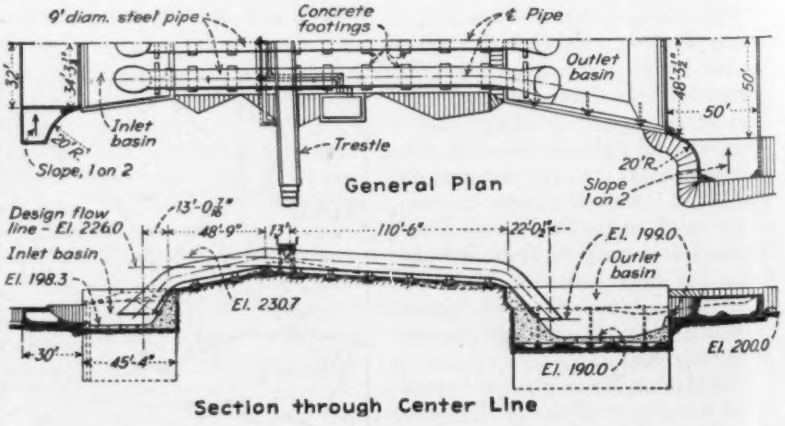


Fig. 4. Plan and section of the siphon installation showing the design of the inlet and outlet basins as well as the flared ends of the tubes.

for the intakes of large capacity, multiple pump units.

Computations based on existing data indicated that a design discharge coefficient of about 0.75 could be expected. With this coefficient and a required discharge of 2,600 cfs, it was determined that three 9-ft. pipes would be required. Subsequent tests (as will be explained later) have shown a coefficient of 0.97 is more nearly the actual value and that the design flow could have been obtained with smaller pipes. However, lack of knowledge concerning losses in pipes under siphonic action required conservative design. It is also true that using the original design data, a greater number of smaller pipes could have been used. While they would have given a somewhat more flexible operation, that advantage would have been offset by the additional cost required to widen the basin and provide more footings.

A requisite to successful construction of the siphon tubes was to produce an airtight job, and electric welding was employed for the fabrication of the tubes. Boiler flange steel was used, which is low in carbon content and therefore easier to weld.

The 9-ft.-diameter siphons are flared at each end with a maximum cross-sectional area at the intake and discharge ends of twice the area of the 9-ft. section. The inlet transition is circular in shape whereas the outlet transition is elliptical and conforms in shape to an ogee section of a spillway; when the siphon operates, turbulence is equally distributed over the basin with a mild back roller. Each welded section is 8 ft. in length and reinforced with structural steel angles to prevent collapse under high vacuum. The pipe is anchored to a rigid foundation at the summit and rests on sliding bearing plates at all other footings to allow it to expand or contract with temperature changes. The ends of the pipes are supported by the back walls of both basins, which are not considered subject to settlement. All intermediate supports are free concrete footings, not supported by piles, placed in the compacted levee; they have not settled under the load.

Outlet basin details

The outlet basin presented problems different from those encountered in the design of the inlet basin. They involved the presence of uplift since the basin is not always submerged, and a determination of dimensions so as (1) to give maximum volume of flow by giving maximum differential in head; (2) to dissipate surplus energy and prevent excessive velocity in channel below; and (3) to provide sufficient water seal to prime one tube when there is no flow through siphon.

Drainage to prevent the build-up of maximum uplift of 12 ft. was provided for by placing a gravel blanket 2 ft. thick under the foundation. The blanket is tapped by six 6-in. pipes extending through the floor of the basin. The basin floor is 10 ft. lower than the outlet sill and insures a true submerged flow (hydraulic jump drowned out). The sill itself is one foot higher than the discharge end of the siphon and this depth of seal provides the necessary volume of water to prime one siphon.

With a depth of basin floor and elevation of sill determined, it next became necessary to insure that the basin had sufficient discharge capacity to allow a free flow from the tubes without building up the lower pool elevation and thereby causing a reduction in effective head producing the flow in the pipes. The maximum width of the basin at the downstream sill is 90 ft. (Fig. 4) which, when using the design flow of 2,600 cfs., gives a mean velocity over this section of 3.5 cfs. Such a width of sill also served to reduce the velocity of water to prevent channel scour.

The problem involved in the actual channel design was twofold. First, it was important to hold the velocities low and prevent scour; and second, it was necessary to keep the velocities sufficiently high to prevent the lower pool elevation building up and reducing the effective differential in head. Both criteria were satisfied by making the channel with a 100-ft. bottom width and 1 on 2 side slopes.

The siphons are primed by a vacuum pump, whose rated capacity is 420 cu. ft. of air per minute, against a vacuum of 20 in. of mercury referred to a 30-in. barometer, and a capacity of 100 cu. ft. of air per minute with a vacuum of 27 in. of mercury. It is of the water-sealed type and is primed by river water provided by a separate pump. It takes about one hour and forty minutes to prime one pipe with a 26-ft. lift. For the more usual lifts the time is much shorter, and at 18 ft. only a half hour is required.

Self-priming experiments conducted on the siphons are of interest. First it should be said that the differential between the two pools during operation of the siphon is a variable quantity, and the siphonic lift is also a variable quantity. At any given lift there is a specific differential between the two pools that gives a

Fig. 5. The siphon structure as it looks from the top of the levee. The discharge is at the right.

velocity through the tube which maintains the prime. At a lesser differential the siphon will lose its prime and at a greater differential the capacity of the flow of water for trapping excess air increases. This capacity for trapping excess air has been used for priming one siphon from the flow of another siphon. When the experiment was carried out it was found that the time required to prime the second siphon from the first was substantially the same as it took to prime the first with the vacuum pump.

Hydraulic design

The size of the siphons proposed made the use of any existing data regarding siphonic action extremely doubtful. Actually, therefore, the hydraulic design became one of a pioneer nature largely based on sound reasoning rather than precedent. After the design was well under way, the writer was happy to obtain the service of A. B. Wood and Wade Barnett of the New Orleans Water Board, who gave very material assistance in executing a successful design.

As has been stated, the discharge was set by the requirements of a navigation permit and so it became necessary to assume certain losses and arrive at a value of head that would produce this flow. Accepted existing formulas for losses due to friction, bends, expansion, intake and exit were used. They gave a coefficient of discharge equal to 0.815 and 0.797 for the center and outside pipes, respectively. These coefficients led to the conclusion that a discharge of 2,600 cfs. could be obtained with the inlet pool at El. 213.6 (mean gulf level) which normally occurs with a tailwater elevation of 208.5 with no backwater effect from a tributary to the St. Francis River above Marked Tree.

Actually, the siphons performed about 20 per cent more efficiently than computed and the design discharge occurs at elevations lower than assumed. It is generally considered that low river stages in the pool begin with El. 210 and that only under special circumstances will the lake level be drawn below this, but the design permits operation down to 207. The intake lip of the siphon is at 203.3, and so adequate provision is made for submergence to prevent intake of air through suckers even at low pool operation. With the inlet pool above El. 213 one or all of the siphons may be operated with a discharge ranging from 800 to 5,000 cfs. The vacuum may be broken by opening the 6-in. valve through the water-sealed centrifugal pump or by opening the 8-in. valve in the summit of each pipe.

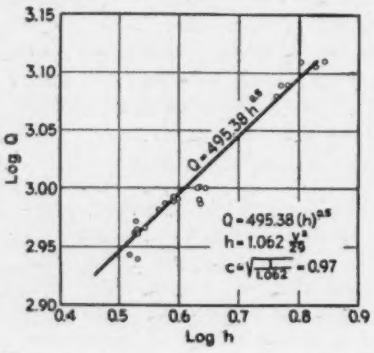


Fig. 6. Discharge data from which the coefficient was computed.

The theoretical lift for mean atmospheric pressure at El. 210 (M.G.L.) is 33.58 ft. of water. If we allow 1.61 ft. due to reduction of barometer at 28.5 in. of mercury at sea level (the lowest reading normally to be expected) and 0.60 ft. for vapor pressure at 60 deg., the net total theoretical lift available is 31.37 ft. The designed normal range of lift is 22 to 28 ft. and is within the theoretical limits of lift and capacity of air pump. Actually, flow has been experienced with a lift of 28 ft., producing a flow of 770 cfs. with one pipe working, and with no tendency for it to lose its prime.

Performance data

Hoping that additional data might be of some value in alleviating design difficulties in future siphon construction a detailed test was conducted on the completed structure. Provisions were made in the siphon tubes for installing piezometer and pitot tubes and accurate discharge measurements were taken at various operating heads. The results of the discharge tests are shown, in part, in Fig. 6.

High velocities and turbulent flow had a pronounced influence on the readings of the piezometers and pitot tubes, and an interpretation of these data was not deemed proper until additional studies have been completed. Discharge measurements were taken well below the outlet end of the siphon, and the flow at the point of measurement was checked by additional measurements over a second range 1,000 feet downstream.

In any experiment of this nature it is impossible to arrive at results that give the same coefficients of discharge for each measurement. To more or less arrive at an average figure for the value, the discharge was plotted against the head. The graph indicated a parabolic function, and in order to facilitate obtaining the equation of this curve, the logarithms of the two functions were plotted and a representative straight line drawn through the points (Fig. 6). The equation of this line was then computed to be $Q = 495.38\ h^{0.5}$. Equating this to $Q = Av$ and solving for H the equation becomes $H = 1.062\ \frac{v^2}{2g}$ and the coefficient of discharge is equal to $\sqrt{1/1.062}$ or 0.97. During this period the heads varied from 3.4 to 7.0 and the lift from a low of 21.8 ft. to a high of 27.0 ft.

This coefficient pertains only to the middle pipe and must be considered as an approximate value. It is probable that the coefficient decreases to some extent with higher heads, and the plotted points in Fig. 6 indicate a slight curve might be more nearly correct that a straight line function. Insufficient data at the higher heads makes it impracticable to attempt comprehensive deductions at present.

Three major factors are probably responsible for this high coefficient of discharge. First, the entire inlet basin was designed to bring the water to the siphon lip and through the transition section with a gradual, accelerated flow, without any abrupt changes in velocity. Second, the entire length of tube was streamlined and conducive to streamlined flow. Third, the outlet basin was designed to have sufficient capacity to let the siphon discharge freely with no tendency for the lower pool to build up.

Further tests on the siphon appear to be justified. Particularly, it would be desirable to know the distribution of losses within the tubes.

The siphon has a distinct place in hydraulic structures. Within limits of siphonic lift it has definite advantage over the culvert type of sluiceway in economy of construction and factor of safety, and the writer believes its use in problems of transporting water across levees, dams, etc., will increase.

FROM FIELD AND OFFICE

Working problems and time-saving methods for engineers and contractors

Filling Holes by Blasting

F. R. COZZENS
Stockport, Ohio

In construction work it often becomes necessary to pull long posts, pillars or piles from the ground, and considerable labor and time are required to refill the holes. Where such holes or cavities exceed 8 ft. in depth, many modern contractors are cutting the task to a minimum by using explosives.

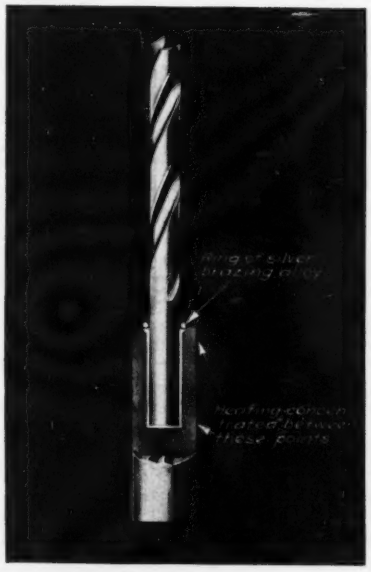
The principle is to drill two holes at an angle, one on either side of the cavity to be filled. The depth of the holes should be one-third the depth of the cavity, and the bottom should reach to within 8 to 10 in. of the cavity's face. Each hole is loaded with two cartridges of dynamite, 40 to 60 per cent strength, stemmed lightly with earth and fired simultaneously. Firing may be done by either fuse or battery.

The force of the blast breaks the cavity face, allowing the crumbled earth to drop in and fill it. Since much of the force is expended through the bottoms of the drill-holes, and through into the cavity, very little spoil is thrown out. The surface, being broken, settles in naturally, and follow-up work is seldom necessary.

• • •

Drill Shank Enlarged By Brazing

Need for larger shanks, $\frac{1}{2}$ to $\frac{5}{8}$ in. on $\frac{31}{64}$ and $\frac{15}{64}$-in. straight-shank drills, in a mid-west plant led to the successful use of brazing as illustrated. New cold-rolled steel shanks of the desired outside diameter were turned and drilled to give 0.002 to 0.003-in. clearance when the old drills were inserted. The old drill shanks and the holes in the new shanks were cleaned and fluxed and the drill inserted in the hole. A ring of silver brazing alloy was then placed around the drill, and the shank was heated with a torch and when a dull cherry red color was reached the silver alloy, which flows freely at 1,175 deg. F., flowed and penetrated between the drill and the new shank.



Method of brazing a small drill into a large shank.

Several completed joints were cut apart and all showed that the brazing alloy had penetrated throughout the joint to the bottom of the hole; the two parts were bonded firmly together. Further tests under severe operating conditions resulted in broken drills repeatedly without disturbing the brazed joints. The heat required reached a dull cherry red, yet by confining it to the shanks and cooling the joint immediately after brazing, the hardness of the cutting end of the drill was not changed.

• • •

Daytime Lighting For Tunnel Entrances

With the building of high speed highways, many of which have tunnels, there is the problem of providing good general lighting as well as safe supplementary daytime lighting for the entrances. The general tunnel lighting should be of the same order as that provided for good street lighting. The accompanying chart, prepared by General Electric engineers at Nela Park, shows the daytime lighting requirements for tunnel entrances. The purpose of this lighting is to give the eyes a chance to readjust themselves to comparative darkness after having been accustomed to several thousand footcandles of daylight. Going into a theatre in the daytime is a familiar experience and illustrates the time element in eye accommodation.

Commonly the supplementary daytime lighting within the entrance is artificial and, as can be seen from line *B* on the chart, a minimum of 100 footcandles should be provided within the entrance and tapered down to two or three footcandles over a distance of 200 ft.

These values are for a driving speed of 35 mph. Where the car speed is greater, the distance over which supplementary lighting is provided should be proportionately greater.

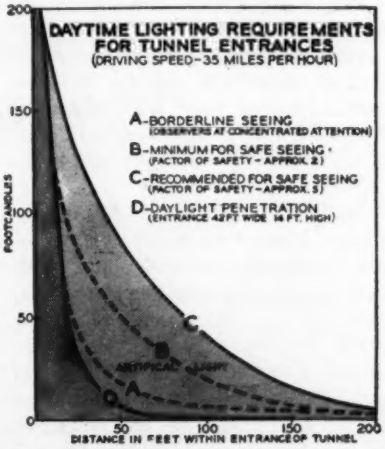


Chart for daytime lighting of tunnel entrances.

For example, for cars traveling 50 mph the supplementary daytime lighting should be provided over 285 ft. to allow the same amount of time for the eyes to become accommodated to the change in lighting values.

BRIEF MEMORANDA

To pull gas from frozen ground after a gas main or service pipe leak or break, the Metropolitan Utilities District, Omaha, Neb., has devised a gas sucker which can be assembled in the shops. This device consists of a small gasoline-driven vacuum pump such as is manufactured by the Roots-Connersville Blower Corp., hose and perforated pipes. The perforated pipes are inserted like wellpoints into bar-holes driven into the ground and sealed at the top with packed dirt.—J. C. DETWEILER, *construction engineer.*

Notes on Sewage Disposal

A monthly summary of current developments conducted by Willem Rudolfs

ANCIENT HISTORY — Langdon Pearse, Chicago, writes anent a note on water-closets (*ENR*, March 14, 1940, p. 398): "It is surprising how the ancient character of such devices has been overlooked—I am informed by my friend, Joshua D'Esposito, that in Herculaneum and Pompeii, the Romans had a system of water carriage for sewage disposal and had water closets, with water flushing, for both men and women. The actual devices have been found in the ruins. It would seem that the Romans had a complete water carriage system for sewage from houses and buildings nearly 2,000 years ago. With the downfall of the Roman Empire and the Middle Ages, sanitation was generally forgotten, and apparently the knights and ladies went back to the old-fashioned dry closets and privies, together with the decadence of the practice of taking baths and the general lack of sanitation. It is strange that this matter has not been commented on more carefully by historians."

Buildings with bathrooms, latrines and drains have been found by excavators dating back 5,000 years; settling of sewage appears to have been practiced near buildings constructed by King Mardukaplamiddina II in Mesopotamia, about 800 B. C.

SLUDGE DISPOSAL—Results on sludge disposal by vacuum filtration and incineration at three cities with new treatment plants are now available for the first time. The largest plant (Chicago) handles a mixture of activated and raw sludge, the second (Minneapolis-St. Paul) disposes of raw sludge and the last (Buffalo) incinerates digested sludge. On account of its low solids content activated sludge requires much more heat for drying than does raw or digested sludge. Mohlman (*Sewage Works Journal*, Nov., 1939) summarizing figures pertaining to the volumes of water to be removed, the chemicals required for dewatering and chemical cost for sludge conditioning, calls attention to the fact that three times as much water must be evaporated at Chicago as at Buffalo to produce one ton of dry sludge.

Some of the pertinent figures on sludge conditioning and filtration allow comparisons of costs.

COMPARATIVE COST DATA

	Sludge	Lb. water per lb. solids	% chemical of dry sludge		Cost per ton of dry solids	
			CaO	$FeCl_3$	Actual	Calculated
Minneapolis	raw	9.7	7.30	2.38	$1.62	$1.29
Buffalo	digested	7.3	9.25	2.31	1.66	1.41
Chicago	act. and raw	33.8	0.00	7.1	2.13	2.13

The actual cost of sludge conditioning is influenced by the local cost of chemicals. If chemical costs were the same at each place the cost of producing a ton of dry solids from a mixture of activated and raw sludge is almost double that of the cost for raw sludge alone—as indicated by the figures in the last column.

MANHOLE AERATION—Natural ventilation of manholes by means of openings in covers increases as the area of such openings in the cover is increased. Air changes are roughly proportional to the area of the openings. Factors such as outside temperature, barometric change, time of year and humidity are of minor importance. Wind velocity or the speed at which air passes across the manhole cover has a marked effect on ventilation, however. Ventilation increases over 50 per cent when the wind velocity increases from 0 to 10 mph.

Experiments with simple vertical ducts placed in the manholes in combination with openings in the covers made by Jones, Miller and Campbell (U. S. Bureau of Mines, *Bull.* R. I. 3496, Feb. 1940) showed that with two 6-in. vertical ducts, one entering near the top and the other at the bottom, 92 air changes a day were obtained even when the manhole had no openings. When the lower duct was used alone 2.3 air changes per day were obtained, but used in conjunction with 6 ventilating openings (0.8 in. diam.) the number of changes was increased to 20; with 48 openings there was 60 times as much ventilation as with the duct alone. It appears that for manholes requiring increased ventilation, where the expense of mechanical ventilation is not warranted, two simple ducts or a lower duct in combination with openings in the cover afford all ventilation necessary at low cost.

FILTERS AND CHROMATES—Interest in the effect of sewage containing chromium plating waste liquors on the efficiency of trickling filters arises from the fact that the chromate present is considered toxic and may have a tendency to accumulate in the filter and gradually retard biological activities. The chief sources of chromium compounds in sewage are from the discharge of waste water from plating processes and from anodizing processes using chromic acid as an electrolyte. Since domestic sewage has a high buffering action it is probable that chromium plate waste liquors would be neutralized before reaching the filters.

Jenkins and Hewitt (*Jour. Society Chemical Industry*, England, Feb. 1940, Vol. 59, p. 41) describe experiments made with potassium chromate at Birmingham. Compared with results obtained by filtering the same sewage without chromate they found that 1 ppm affected somewhat nitrification and purification of organic matter, while 10 ppm chromium reduced nitrification and oxidation appreciably, sufficient to convert a good effluent into one of only moderately good quality. With 100 ppm nitrification was reduced about 70 per cent with a poor quality effluent. The upper part of the filter retained about twice as much deposited solid matter, resulting in ponding. Interaction between sewage solids and chromium plating wastes may cause deposition of solids containing chromium in sewers.

LARVAE of the filter flies and small worms are effective in scouring, prevent accumulation of growth and debris, aid in unloading and prevention of pooling and choking of the beds.

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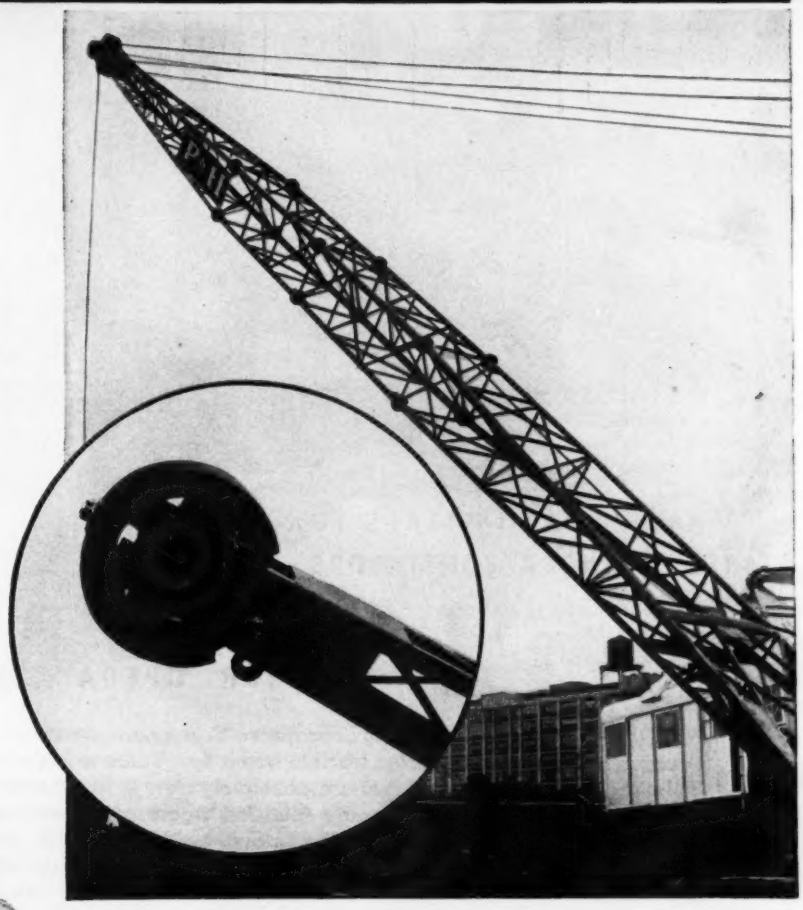

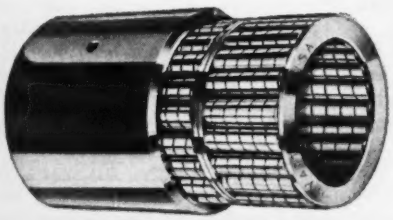

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MEN AND JOBS

E. A. Wood, city planning engineer of Dallas, Tex., working in collaboration with Hare & Hare, Kansas City, Mo., is outlining a park system of boulevards for the city of Dallas.

Edward R. Herman has begun new duties as director of engineering operations in the Williamsport, Pa. district of WPA to succeed Daniel R. Mehl, who was recently promoted to manager of the Williamsport district. Herman has been a member of the WPA office staff at Harrisburg since 1935. Philip Mathews is state WPA administrator for Pennsylvania.

A. H. Horton, district engineer, is in charge of the Middle Atlantic States district office of the Water Resources branch of the U. S. Geological Survey located in the new engineering building of the University of Maryland since June 5.

J. C. Gabbard, director of district operation for the WPA at Tulsa, Okla., has been appointed assistant state operations director for the WPA state office with headquarters at Oklahoma City. Gabbard will be succeeded by J. L. Forbis, former district planning engineer for the WPA at Oklahoma City.

Paul F. Helm, who has been in charge of all engineering of the Indianapolis Power & Light Co., June 5 became assistant to Dan C. Hess, vice president in charge of operation of the utility. Helm attended Purdue University.

A. A. Rhomberg, city manager of Dubuque, Iowa, has been instructed by the Dubuque city council to study the site for a proposed bridge across the Mississippi River at Dubuque.

Thomas Crist, William Pospick and Oliver G. Rayner, who received civil engineering degrees from Southern Methodist University early in June, have all obtained positions with the Texas State Highway Department. Pospick will work in the laboratory for materials and tests at Austin, and Rayner will be junior resident engineer at Atlanta, Texas. Crist's location has not been announced.

Adolphus Mitchell, Kinston, N. C., on June 1 began new duties as traffic engineer for the North Carolina Highway and Public Works Commission. Mitchell, who holds a degree in civil engineering and a master of science de-

(Continued on p. 86)

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CONTRACT UNIT PRICES

What contractors are bidding on different kinds of construction work

KANOPOLIS DAM TUNNEL KANSAS

OWNER: U. S. Engineer Office, Kansas City, Mo.

PROJECT: Construction of outlet tunnel and complete accessories for Kanopolis Dam on the Smoky Hill River, 18 miles southeast of Ellsworth, Ellsworth County, Kan. Involves construction of 2,300-ft. concrete lined tunnel, reinforced with bars, ring beams and purlins. Inside diameter of tunnel 14 ft., 18 in. concrete lining. Temporary buildings and access roadway also to be built. This contract includes open-cut and tunnel excavation and backfill; steel bracing for tunnel and open-cut excavations; concrete for tunnel lining, intake tower, operating house, approach channel, stilling basin, and service bridge; steel for trash racks and service bridge; and control gates complete.

CONDITIONS: All materials to be furnished by contractor, and work must be completed in 500 calendar days. Railroad transportation to Langley, 5 miles from dam site. Highway facilities also available. Tunnel to be constructed through shale. Other contracts on project may be let while contractor is prosecuting this one. Wage rates are: skilled, $1.00 to $1.12½ per hr.; semi-skilled, 55 to 75c.; and common, 40c.

BIDS: Nineteen bids were received May 10, 1940, ranging from the contract low of $793,124 to $1,154,014.

LIST OF BIDDERS:

1. Morrison-Knudsen Co., Inc., Boise, Idaho (contract) $793,124
2. John Kerns Constr. Co., Omaha, Neb. 814,520
3. The Utah Constr. Co., San Francisco, Calif. 856,519
4. Leo Sanders, Oklahoma City, Okla. 860,064
5. A. Guthrie & Co., Inc., St. Paul, Minn. 880,176
6. Missouri Valley Bridge & Iron Co., and Winston Bros. Co., Leavenworth, Kan. 883,793
7. J. A. Terteling & Sons, Boise, Idaho 896,361
8. J. A. Tobin Constr. Co., Kansas City, Kan. 898,102
9. Massman Constr. Co., Kansas City, Mo. 900,286
10. Drainage Contractors, Inc., Detroit, and Nicholas D. Mancini, Grosse Pointe Park, Mich. 929,000
11. Frank J. Knight Co., and Bass Engr. & Constr. Co., Detroit, Mich. 944,007
12. Geo. W. Condon Co., and Pioneer Constr. & Engr. Co., Omaha, Neb. 947,564
13. Gordon Constr. Co., Denver, Colo. 956,942
14. Peter Kiewit Sons' Co., Omaha, Neb. 978,000
15. Forcum-James Co., Dyersburg, Tenn., and Clark, Kearney & Stark, St. Louis, Mo. 1,018,196
16. Ed. H. Honnen Constr. Co., Colorado Springs, Colo. 1,067,858
17. W. E. Kier Constr. Co., San Diego, Calif. 1,120,965
18. Marsch Constr. Co., Chicago, Ill. 1,140,090
19. Knutson Brothers, Kansas City, Mo. 1,154,014

Item	Quan.	Unit Prices (1)	(2)	(3)
1. Open-cut excavation	230,000 c.y.	$0.30	$0.35	$0.30
2. Tunnel excavation	23,100 c.y.	4.50	4.00	5.75
3. Liquid sealing solution (each coat)	6,100 sq.	1.00	3.00	1.25
4. Backfill and embank., excav. mat'l	15,000 c.y.	.20	.20	.22
5. Backfill and embank., borrow pit mat'l	6,000 c.y.	.30	.32	.30
6. Gravel drains	100 c.y.	3.00	4.00	3.55
7. Rock paving	170 sq.	30.00	25.00	34.00
8. Grout for rock paving	10,000 c.f.	.50	.30	.40
9. Wr. iron sealing strips	42,000 lb.	.13	.10	1.25
10. Joint filler	100 s.f.	1.00	1.00	.50
11. Reinforcing steel	2,000,000 lb.	.039	.04	.0405
12. Wire fabric	1,000 lb.	.10	.16	.10
13. Cl. A conc., tunnel lining	9,070 c.y.	11.20	12.00	14.30
14. Cl. A conc., intake tower	4,140 c.y.	14.00	15.00	15.30
15. Cl. B conc., approach channel	1,720 c.y.	11.00	12.00	11.70
16. Cl. B conc., stilling basin	6,050 c.y.	10.00	12.00	11.70
17. Cl. A conc., oper. house and stairs	150 c.y.	30.00	18.00	40.00
18. Cl. B. conc. serv. br. piers and abut.	330 c.y.	11.00	14.00	12.50
19. Cl. A. conc., serv. br. floor slab	50 c.y.	30.00	15.00	26.50
20. Gunite coating (each coat)	100 sq.	5.00	10.00	6.00
21. Grout holes	2,300 l.f.	.40	.75	1.00
22. Pres. grout of neat cement	5,000 c.f.	1.35	1.10	1.30
23. Pres. grout cement and sand	5,000 c.f.	1.35	.80	1.10
24. Drill holes for slab anchors	1,400 l.f.	.40	.40	.55
25. Grout for slab anchors	60 c.f.	1.00	1.10	1.65
26. Oper. house, except conc. and reinf.	L.S.	1,800.00	1,200.00	2,000.00
27. Steel bracing, open-cut excav.	150,000 lb.	.075	.05	.058
28. Steel bracing, tunnel excav.	1,185,000 lb.	.062	.07	.069
29. Trash racks	94,000 lb.	.075	.05	.0675
30. Service bridge	141,200 lb.	.075	.06	.075
31. Misc. metal parts	38,600 lb.	.15	.12	.105
32. Wr. iron hand rails	5,000 lb.	.30	.20	.24
33. Equipt. intake tower and oper. house	L.S.	89,500.00	100,765.00	86,000.00
34. Field office, oil storage, garage	L.S.	7,000.00	8,000.00	12,200.00
35. Paving access road	8,900 s.y.	1.70	1.50	1.40
36. Excav. for access road	9,000 c.y.	.20	.18	.30

BRIDGE AND APPROACHES CONNECTICUT

OWNER: Connecticut State Highway Dept., Hartford; Wm. J. Cox, commissioner; Leslie G. Sumner, bridge engineer.

PROJECT: Three span deck girder, reinforced concrete bridge and approaches over Shetucket River in the town of Sprague, Conn., north of Norwich. Two end spans 76½ ft. long; center span 87 ft. long. Distance from bottom of concrete pile cap to roadway 25 ft. End spans cantilever 13½ ft. beyond piers and support suspended span by end links.

CONDITIONS: Shetucket River subject to "flash" flows. Traffic must be maintained. Old superstructure damaged by floods to be removed. Contractor to furnish all materials except cement, which will be furnished by state at $2.50 per bbl. Rail and highway transportation facilities available, and project to be finished in 210 calendar days. Wages are: skilled labor, 75c. to $1.65 per hr.; semi-skilled, 60 to 80c.; and common labor, 50c. per hr.

BIDS: Six bids were opened May 13, 1940, ranging from the contract low of $107,093 to $118,943.

LOW BIDDERS:

1. Alexander Jarvis Co., Manchester, Conn. $107,093
2. Frank T. Westcott, No. Attleboro, Mass. 110,843
3. Mariani Constr. Co., New Haven, Conn. 113,336

Item	Quan.	Unit Prices (1)	(2)	(3)
1. Clear and grub	L.S.	$1,500.00	$200.00	$1,000.00
2. Unclassified excavation	757 c.y.	.40	2.50	.50
3. Borrow	7,490 c.y.	.25	.30	.30
4. Formation of subgrade	2,568 s.y.	.10	.10	.10
5. Shape slope and shoulders	750 l.f.	.50	.20	.30
6. Bridge excavation	2,200 c.y.	1.50	2.50	6.00
7. Gravel fill subbase	350 c.y.	.40	.60	1.00
8. Broken stone (base)	761 ton	1.75	2.50	1.75
9. Broken stone (bit.mac.surf.)	310 ton	2.40	3.00	2.50
10. Bit. mat'l., asph. for penetr.	4,948 gal.	.10	.10	.12
11. Bit. mat'l., asph. surf. trtmt	819 gal.	.10	.10	.15
12. Class A. concrete	1,206 c.y.	12.00	15.00	15.00
13. ½-in. cork or rubber exp. jt. filler	244 s.f.	.30	.50	.50
14. Deformed steel bars	101,611 lb.	.04	.05	.04
15. Steel fabric (2- x 3-in. x # 12)	3,300 s.y.	.40	.50	.60
16. Structural steel	490,900 lb.	.045	.045	.04
17. Steel castings	8,500 lb.	.22	.20	.15
18. Copper flashing	45 lb.	.60	.50	1.00
19. Steel bearing piles	813,400 lb.	.03	.027	.0275
20. Drive steel piles	14,625 l.f.	.35	.30	.30
21. Loading test piles	2 ea.	50.00	200.00	100.00
22. Equip. for driving test piles	L.S.	10.00	500.00	150.00
23. Riprap	1,185 ton	.50	3.00	2.00
24. Waterproof painting	355 s.y.	.35	.25	.30
25. Iron bridge rail	486 l.f.	7.00	6.00	5.00
26. Wire rope railing	640 l.f.	.45	.50	.40
27. Type B anchorages	4 ea.	12.00	15.00	10.00
28. Single posts	20 ea.	1.65	2.00	2.00
29. Remove old superstructure	L.S.	400.00	100.00	400.00
30. Type C anchorages	2 ea.	8.00	8.00	10.00
31. Compensating type anchorages	6 ea.	7.00	10.00	10.00
32. Cement mortar surfacing	23,500 s.f.	.50	.30	.35
33. Loose gravel surface	10 c.y.	1.00	2.00	1.00
34. Rolled bank run gravel surf.	170 c.y.	1.00	1.00	1.50
35. Sand cover	691 s.y.	.05	.10	.05
36. Portland cement	2,250 bbl.	2.50	2.50	2.50
37. Broken stone, drains	305 ton	2.00	2.50	2.00
38. Relocate wood steps	L.S.	30.00	150.00	100.00

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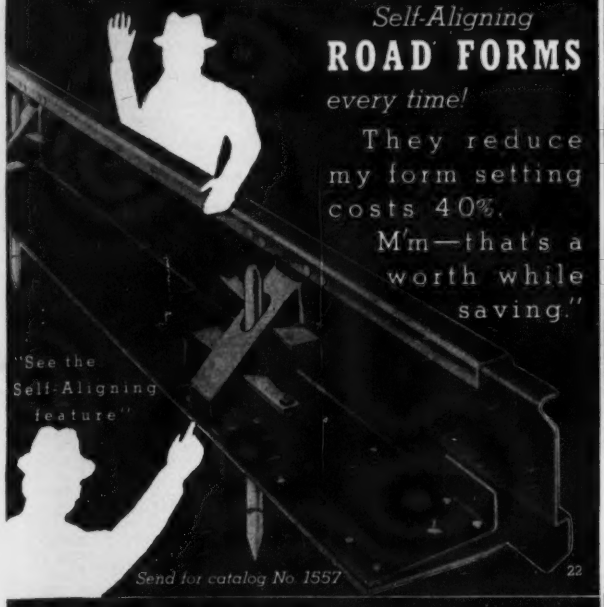

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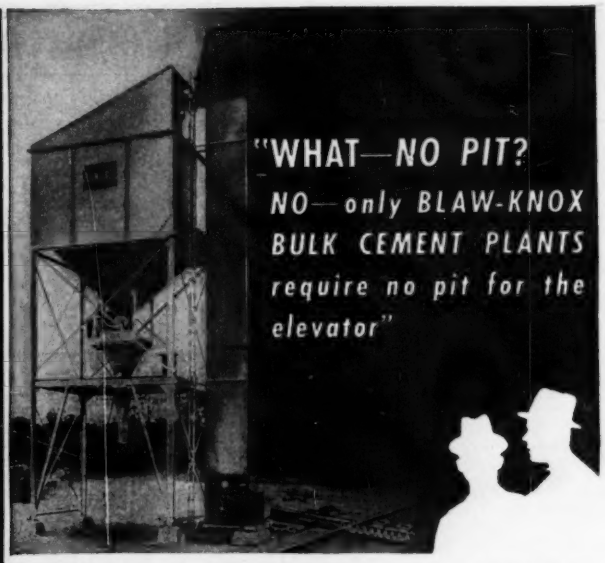

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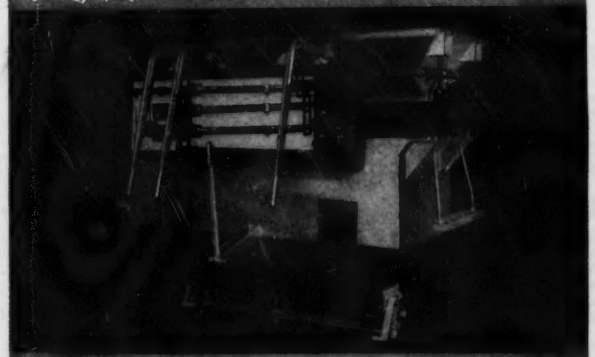

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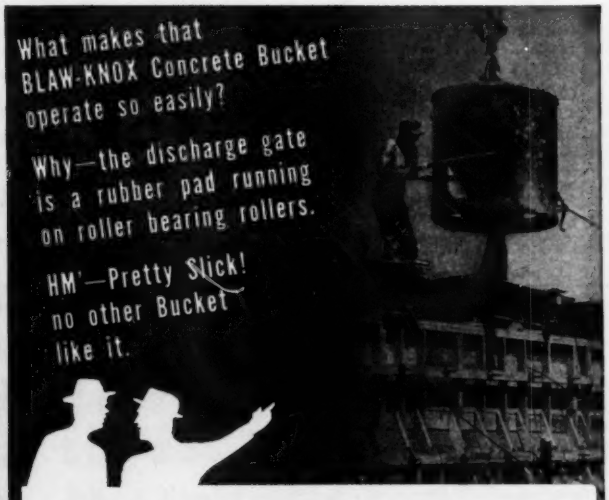

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BLAW-KNOX *Roller Gate* CONCRETE BUCKETS

MATERIALS AND LABOR PRICES

Market quotations on construction materials and wage rates reported monthly by ENR correspondents

LUMBER AND LABOR provide the most important price changes in July. Lumber prices are mixed. Short Leaf Yellow Pine is up 50c. to $1.00 on most sizes in Atlanta; up $1.00 on 1x8-in. size in Chicago; up $1.00 to $2.00 in Cleveland; but off $2.00 to $5.50 in Baltimore; off 50c. to $3.00 in Birmingham; and 50c. to $1.00 in St. Louis. Long Leaf Yellow Pine is $2.00 to $5.00 lower in Birmingham. Douglas Fir is down $1.00 on 2x8-in. size in Baltimore, but up $1.00 on 1x8-in. in Chicago. Plywood base price at Seattle remains unchanged.

Cement is unchanged in the 20-cities' list. Ready-mixed concrete is down 40c. per cu. yd. in Kansas City, and off 25c. in New York. Reinforcing bars are mixed, up 30c. per 100 lb. in Detroit, but 25c. lower in Minneapolis and St. Louis, and 20c. lower in Seattle. Common brick price is up 50c. per M in Los Angeles, the only change in the list.

Carpenters' rate in New Orleans increased 5c. per hr. to $1.25. Structural ironworkers' rates rose 12½c. per hr. in Boston and Detroit, 10c. in San Francisco, 7½c. in New York, but are 12½c. lower in New Orleans. Hoisting engineers' wage ranges from $1.37½ to $1.62½ per hr. in Boston, whereas formerly the top rate prevailed. The engineers' rate is down 12½c. in New Orleans.

Common labor on heavy construction in Detroit now ranges from 60 to 77½c., a slight increase from the former 60 to 75c. quotation. The New Orleans common labor rate is now 55c. per hr., 15c. higher than a month ago.

CEMENT, AGGREGATES, READY-MIXED CONCRETE—F.O.B. CITY

	PORTLAND CEMENT — Per bbl., carload lots, including 40c per bbl. for bags, cash discount not included			SAND AND GRAVEL — Per ton, carload lots			CRUSHED STONE — Per ton, carload lots		CRUSHED SLAG — Per ton, carload lots, f.o.b. plant		CONCRETE Ready Mixed 1:2:4, 50 c.y. or more, delivered
	Cloth Bags	Paper Bags	Bulk	Gravel, 1½ in.	Gravel, ¾ in.	Sand	1½ in.	¾ in.	1½ in.	¾ in.	
Atlanta	$2.90	$2.66	$2.61	$1.75	$1.85	$1.95	$1.65	$1.75	$1.55	$1.65	$7.90
Baltimore	2.31	2.06	1.86	1.70‡	1.70‡	1.20‡	1.75‡	1.75‡	1.75‡	1.75‡	6.50
Birmingham	2.50	2.25	2.05	1.75	1.75	1.50	1.00	1.00	.90	1.15	5.80
Boston	2.75*b*	2.50*b*	2.30	1.10‡	1.10‡	.80‡	1.10‡	1.10‡	....	....	6.75*m*
Chicago	2.60	2.35	2.20	2.00‡*d*	2.00‡*d*	2.00‡*d*	2.00‡*d*	2.00‡*d*	1.25‡	1.25‡	....
Cincinnati	2.58	2.33	2.08	1.20	1.20	1.10	1.80	1.80	....	....	6.60
Cleveland	2.00	1.75	....	1.45*i*	1.45*i*	1.30*i*	1.70*i*	1.70*i*	1.70*i*	1.70*i*	6.55*i*
Dallas	2.30	2.05	1.85	2.00*p*	2.00*p*	1.50*p*	1.09*p*	1.84*p*	....	....	6.20
Denver	3.00	2.75	....	1.20	1.40	.90	....	....	....	....	7.25
Detroit	2.23	1.98	1.78	1.30	1.40	.80	1.30	1.45	1.02‡	1.14‡	6.50
Kansas City	2.56	2.31	2.16	1.85	2.30	1.00	1.91	1.91	....	....	7.50*n*
Los Angeles	2.25*b*	2.05*b*	....	1.00	1.00	1.00	1.00*f*	1.00*f*	....	....	5.85
Minneapolis	2.71	2.46	....	1.00*h*	1.00*h*	.25*h*	1.00*h*	1.00*h*	....	....	7.35*j*
Montreal	....	1.75*r*	1.60*r*	....	....	1.25‡	.85‡*c*	1.00‡*c*	....	....	6.45*i*
New Orleans	2.41	2.16	1.96	1.50	1.50	1.25	....	....	....	....	7.25
New York	2.60‡	2.35‡	....	1.40*de*	1.40*de*	.90*de*	1.65*de*	1.75*de*	....	....	6.75
Philadelphia	2.32	2.07	1.87	1.55‡	1.65‡	1.25‡	....	....	1.00	1.00	7.95*k*
Pittsburgh	2.44	2.19	1.99	1.40‡	1.40‡	1.70‡	2.25‡	2.25‡	1.50	1.50	7.50
St. Louis	2.55	2.30	....	1.50‡	1.50‡	1.25/1.50‡	.85*h*	.85*h*	.75*g*	.75*g*	7.50
San Francisco	2.16	1.96	1.66	1.36	1.36	1.36	1.46	1.46	....	....	6.95
Seattle	2.95*o*	2.70*o*	....	1.00*o*	1.00*o*	1.00*o*	2.00*o*	2.00*o*	....	....	7.25

‡ Delivered. *a* 10c. allowed for each returnable bag. *b* 10c. per bbl. off for cash. *c* Plus municipal tax. *d* Per cu. yd. *e* Barge lots alongside docks *f* Crushed granite. *g* F.o.b. Granite City, Ill. *h* F.o.b. plant. *i* Within three miles of Public Square. *j* 5% discount for cash. *k* Discount 35c. 500 to 2000; 70c. 2000 to 5000; 95c. over 5000. *l* Up to 200 cu. yd. *m* 50c. off for cash. *n* 25 c.y. or more. *o* 2% off for cash. *p* 10c. per ton off, cash 15 days. *r* 10c. per bbl. off, cash 20 days.

CASH DISCOUNTS CEMENT to Contractors: truck delivery, 2% for cash on 10th of month; on carload deliveries, no trucking, same as dealer cash discount 10c. per bbl. for payment within 15 days of date of invoice. Subject discount 10c. per bbl. 20 days in Montreal.

CURRENT MAXIMUM PRICES RECEIVED AT CEMENT MILL

Charge for bags not included. For cloth bags, add 40c. per bbl.; 10c. refund allowed for each returnable bag; for paper bags add 15c. per bbl., not refundable

	Bagged	Bulk		Bagged	Bulk		Bagged	Bulk
Buffington, Ind	$1.70	$1.65	Ironton, Ohio	$1.60	$1.55	Richard City, Tenn	$1.80	$1.75
Dallas, Tex. (Inc. 5c. tax)	1.80	1.75	Limedale, Ind	1.70	1.65	Steelton, Minn	1.80	1.75
Hannibal, Mo	1.70	1.65	Norfolk, Va	1.65	1.60	Universal, Pa	1.65	1.60
Hudson, N. Y	1.75	1.70	Northampton, Pa	1.55	1.50	Waco, Tex. (5c. Tax inTex.)	1.75	1.70
Independence, Kans	1.70	1.65	North Birmingham, Ala	1.70	1.65	Montreal	1.55	1.55

PAVING BRICK, BLOCKS, ASPHALT, ROAD OILS—F.O.B. CITY

	PAVING BRICKS AND BLOCKS — Granite per M. lots of 50,000 4x4x8 in.	Brick per M. 3x4x8½ in. carload lots	Wood per sq. yd. 3½ in. 16-lb. treat	PAVING ASPHALT — Per ton, less than 80 penetration: Tank car	Drums	ASPHALT BINDERS — FLUXES — Per gal. 80–300 penetration: Tank car	Drums	CUTBACK ASPHALT — Per ton: Tank car	Drums	ROAD OILS — Per gal.: Tank car	ASPHALT EMULSION (Quick-breaking) Per gal.: Tank car	Drums
Atlanta	$73.00	$39.35	$2.25	$16.60*e*	$22.83*e*	$0.0645*e*	$0.1165*e*	$0.0684*he*	$0.1207*h*	$0.0634*e*	$0.0644	$0.0684
Baltimore	140.00‡	44.00‡	2.75	14.00	20.00	.06	.09	.065*h*	.115*h*	.....	.....	.....
Birmingham	.....	27.50*j*‡	.....	18.20	23.20	.0747	.1047	.0834*h*	.1484*h*	.....	.....	.....
Boston	90.00	47.00	3.25	14.00	20.00	.06	.09	.065*h*	.115*h*	.....	.09‡	.12‡
Chicago	140.00	49.00	1.80*d*	14.50	18.50	13.00*g*	17.00*g*	.06*h*	.095*h*	.04	.09	.125
Cincinnati	112.00	37.50	.....	14.31	18.31	.10	.....	18.00	.....	.0525	.....	.08
Cleveland	110.00	37.50	2.80	14.50	19.50	.065	.075	.075*h*	.085*h*	.0375	.08	.09
Dallas	.....	30.00	.....	13.25	20.30	.06	.103	15.00	24.80	.055	.05/.07	.07/.09
Denver	.....	27.50	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Detroit	.....	35.00	.....	13.15*f*	18.45*f*	.061	.0984	13.15	17.20	.0475	.061	.0923
Kansas City	.....	44.00	.....	18.50	23.50	.077	.1075	.077*h*	.107*h*	.057	.11	.15
Los Angeles	.....	50.00	.....	6.50*b*	15.00	6.50*bg*	15.00*bg*	5.50*b*	.....	.0192	.0525	.1125
Minneapolis	.....	.....	1.90*d*	19.20	24.20	.10	.12	19.30	28.30	.056	.....	.....
Montreal	75.00	.....	3.62	14.60	21.25	.105	.1375	.101*h*	.121*h*	.07	.135	.188
New Orleans	.....	.....	.....	14.00*a*	19.00*a*	.055	.085	.06*h*	.09*h*	.....	.078	.....
New York	.....	.....	2.75	15.00	21.00	.06	.09	.065*h*	.115*h*	.....	.07	.11
Philadelphia	120.00	42.50	2.84	14.00	19.00	.055	.08	.06*h*	.09*h*	.05	.06	.09
Pittsburgh	135.00	38.75	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
St. Louis	110.00/121.00	37.00	.....	14.00	19.00	14.00*g*	19.00*g*	.06*h*	.095*h*	.04	.09	.125
San Francisco	.....	70.00	.....	6.50	15.00	6.50*g*	15.00*g*	7.00	17.50	.025	.0525	.1125
Seattle	.....	60.00*c*	.....	14.70	22.90	.....	23.50*g*	9.50/10.50	23.00/24.00	7.75*g*	.0575*i*	.1308*i*

‡ Delivered. *a* F.o.b. Baton Rouge. *b* Delivered to purchaser's warehouse. *c* 3½x4x8½ in. *d* 2½ in. 6 to 8-lb. treatment. *e* Local reduction due to 20% reduction intra-state class freight rates, only Georgia affected. *f* Mexican. *g* Per ton. *h* Per gallon. *i* F.o.b. Martinez. *j* 3x3½x8½ in.

(Continued on page 80)

Over a Billion Yards of Experience In Bucyrus-Erie Hydraulic Dredges

Built in a wide range of sizes, to handle a wide range of materials, Bucyrus-Erie hydraulic dredges have moved over a billion yards. This unequalled experience is built into each modern dredge and is assurance of proven performance in the Bucyrus-Erie dredge you buy. Designed by men who know the problems through experience; built by skilled specialists with the most modern, complete facilities in the world; "tailored" to meet YOUR problems, Bucyrus-Erie dredges have thoroughly proved their mettle. It will pay you to investigate.

Bucyrus·Erie

SOUTH MILWAUKEE, WISCONSIN

IRON AND STEEL PRODUCTS—BASE MILL PRICES

	STRUCT. SHAPES-PLATE	REINF. BARS ½-in. billet *b*	RIVETS ½-in. structural	WIRE NAILS Base *d*	SHEET PILING Base	STEEL RAILS Per Gross Ton: Standard	Light	Re-rolled	TRACK SUPPLIES: Angle Bars	Std. Spikes *c*	Tie Plates *c*	Track Bolts
Birmingham	$2.10	$1.90	$3.40	$2.55	....	$40.00	$40.00	$39.00	$2.70	$3.00	$2.15	$4.15
Chicago	2.10	1.90	3.40	2.55	$2.40	40.00	40.00	39.00	2.70	3.00	2.15	4.15
Pittsburgh	2.10	1.90	3.40	2.55	2.40	40.00	40.00	39.00	2.70	3.00	2.15	4.15
Buffalo	2.10	1.90	....	....	2.40	....	....	....	....	....	2.15	....
Cleveland	2.10	1.90	3.40	2.55	....	....	....	....	....	....	....	....
Youngstown	....	1.90	....	....	....	....	....	....	....	3.00	....	....
Detroit	....	2.00‡	....	....	....	....	....	....	....	....	....	....
Gulf ports	2.45*a*	2.25*a*	....	....	2.85*a*	....	....	....	....	....	....	....
Pacific ports	2.75*a*	2.30*a*	....	....	2.95*a*	....	....	....	....	3.00	2.30	....

‡ Delivered. *a* F.o.b. cars dock. *b* Rail steel same as billet prices. *c* Other basing points include Portsmouth, O., Weirton, W. Va., St. Louis, Kansas City, Minnequa, Colo., and Pacific coast ports, on tie plates alone, Steelton, Pa., on spikes alone, Lebanon, Pa., Richmond, Va. *d* Add switching charge $18 per c.l.

IRON AND STEEL PRODUCTS—F.O.B. WAREHOUSE, PER 100 LB., BASE PRICE

	STRUCTURAL SHAPES Per 100 lb., base price	REINFORCING BARS* Per 100 lb., ½ in., base price: 15 tons or over *b* New billet	Rail steel	Add Switch	$/cwt. for Del.	EXPANDED METAL LATH—Per 100 sq.yd., carload lots: Std. diamond mesh, 3.4 lb.	Std. ribbed 3.4 lb.	WELDED FABRIC REINFORCING—Per 100 s.f., carload lots: 4x16 in., No. 5 & 10 wires	4x12 in., No. 8 & 12 wires	6x6 in. No. 6 & 6 wires Per sq. yd.	SHEET PILING Per 100 lb. base price
Atlanta	$2.34	$2.39	$2.14	.02½	.10	$21.00	$23.00	$1.82	$1.35	$0.1719	$3.04
Baltimore	3.60	2.85	2.70	.....	.10	23.00	25.00	1.70	1.27	.1611	2.80
Birmingham	2.10*a*	1.90	1.90	.02½	.10	20.50	22.50	1.80	1.34	.1701	....
Boston	3.85	2.61	2.46	$20 C.L.	.10	21.50	23.50	1.76	1.31	.1656	3.14
Chicago	2.10*a*	1.90	1.90	.03	.05	19.50	21.50	1.67	1.25	.1575	2.40
Cincinnati	3.68	2.41*c*	2.41*c*	.....	.05	20.50	22.50	1.66	1.25	.1566	....
Cleveland	3.58*g*	1.90	1.90	.02½	.10	20.50	22.50	1.66	1.25	.1566	....
Dallas	4.75	3.25	3.25	.....	.....	25.00	27.00	1.93	1.42	.1827	....
Denver	4.35	3.95	3.80	.....	.....	23.50	25.50	1.97	1.45	.1863	....
Detroit	3.60‡	2.35	2.20	.....	.10	22.50	24.50	1.68	1.26	.1584	2.66
Kansas City	4.00	2.51*c*	2.36*c*	.0	.05	18.50	20.50	1.78	1.33	.1683	2.86
Los Angeles	3.65*f*	2.25	2.10	.02½	.05	23.00	27.50	1.93	1.42	.1827	3.65
Minneapolis	3.80‡	2.23*c*	2.13*c*	.0	.10	20.00/21.50	23.00/24.50	1.80	1.34	.1701	2.73*d*
Montreal	3.90	2.70	2.60	.....	.....	26.50	29.00	2.99	2.08	.319	....
New Orleans	4.40	2.52	2.52	.05	.10	21.50	23.50	1.82	1.35	.1719	2.875
New York	3.75	2.39	2.39	.....	.10	20.50	22.50	1.76	1.32	.1665	2.74
Philadelphia	3.55	2.66	2.56	.....	.....	19.50	21.50	1.71	1.28	.1611	2.72
Pittsburgh	2.10*a*	1.90	1.90	.02½	.10	21.50	24.50	1.59	1.20	.1503	2.40
St. Louis	3.47	2.14	2.14	.0	.05	24.00	28.00	1.69	1.27	.1602	2.70
San Francisco	3.35	2.27*5*	2.125	.02½	.10	19.00	25.50	1.93	1.42	.1827	3.25
Seattle	2.75*e*	2.30*e*	.....	.....	.....	25.50	27.50	1.93	1.42	.1827	2.95*e*

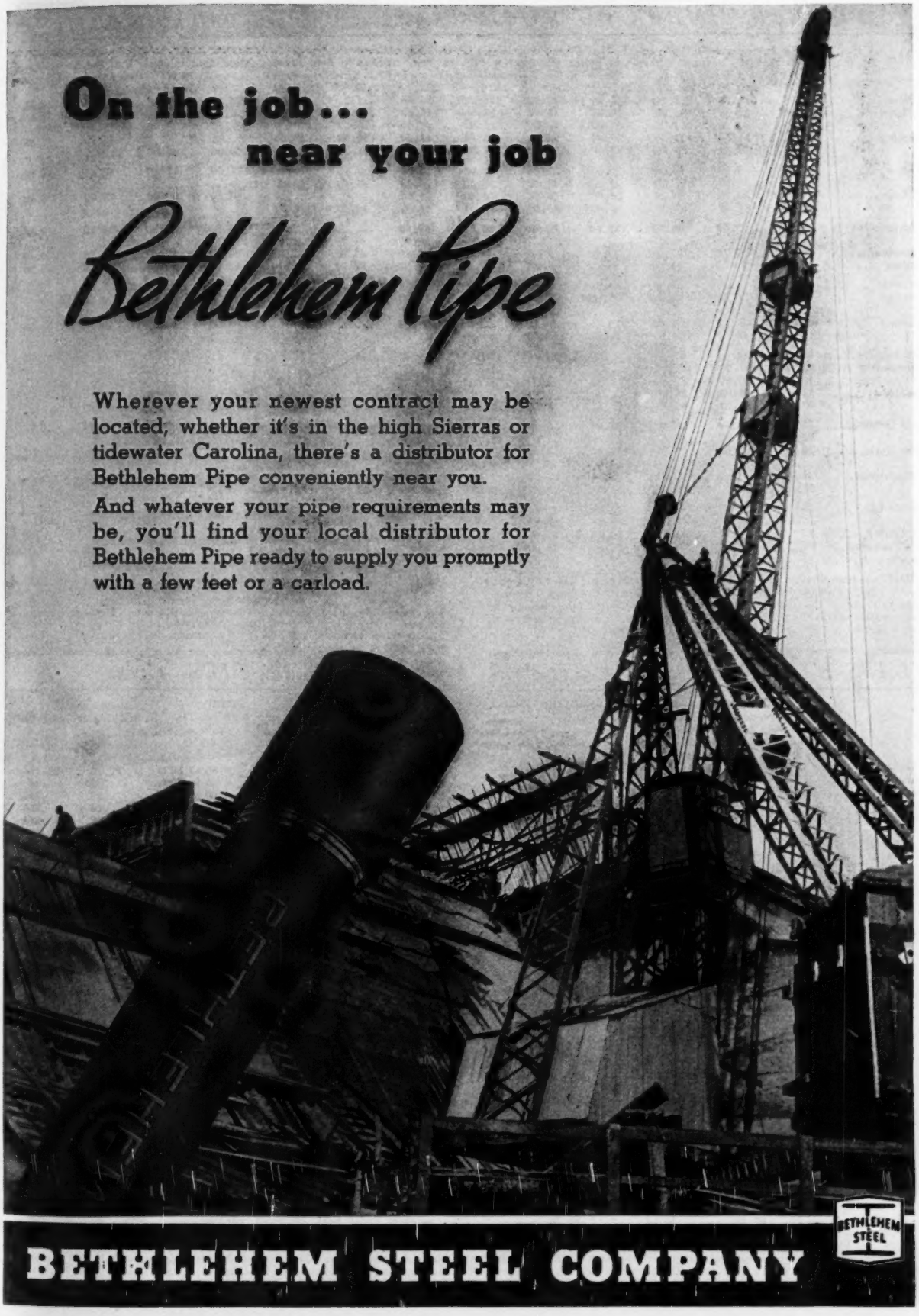
‡ Delivered. *a* Mill prices. *b* 5–15 tons, add 15c. 1–5 tons, add 25c. Less than 1 ton, add 50c. *c* 20 tons or over Base. *d* Mill price plus freight. *e* F.o.b. dock. *f* Includes delivery in free delivery zone. *g* Less than 1 ton, add 30c.; 1 to 5 tons, add 10c. * High scrap steel prices cut former 15c. differences between new billet and rail steel in many mills.

PLUMBING, HEATING, WATER, SEWER AND DRAIN PIPE

	C. I. PIPE Per net ton f.o.b. 6 in. to 24 in.*a*	VITRIFIED SEWER PIPE Per foot, std. 8 in., s.s.	delivered std. 12 in., s.s.	ASTM 24 in.,	C13–35 36 in.,	CLAY DRAIN TILE Per 1,000 ft., carload lots, 6 in.	f.o.b. 8 in.	CONCRETE SEWER PIPE Per ft., delivered: ASTM 12 in.	C 14–35 24 in.	WROUGHT STEEL PIPE Full standard weight, *h* 1 to 3 in., Black %	Butt Weld 3½ to 6 in., Lap Weld Galv. %	Black %	Galv. %
Atlanta	$49.40	$0.268*c*	$0.468*bc*	$1.8525	$4.68*c*	$85.00	$128.00	$0.45	$1.642	62.2	54.2	59.7	51.2
Baltimore	52.80	.30	.55	2.30	6.80	110.00	150.00	.50	1.75	65.5	57.5	63.0	54.5
Birmingham	45.00	.26	.468	1.7875	4.66	85.00	220.00	.40	1.60	62.2	54.2	59.7	51.2
Boston	55.00	.231	.4455	1.89	5.2275	180.00‡	280.00‡	.40	2.00	64.1	56.1	61.6	53.1
Chicago	53.80	.28	.54	2.25	6.25	100.00‡	150.00‡	.38	1.48	67.0	59.0	63.5	55.0
Cincinnati	52.70	.245	.4995	1.9335	5.6375	105.00	170.00	.60	1.70	65.9	57.9	63.4	54.9
Cleveland	54.24	.196	.378	1.485	4.51	69.00	108.00	.35	1.50	67.8	59.8	65.3	56.8
Dallas	58.20	.265	.477	1.728	4.70	80.00	100.00	.46	1.57	59.9	51.9	57.3	48.3
Denver	60.60	.24†	.52†	2.16†	.....	130.00	240.00	.....	.....	58.4	50.4	55.7	47.2
Detroit	54.24	.245	.4725	1.935	4.408	66.45	105.00	.33	1.50	66.3	58.3	63.8	55.3
Kansas City	55.12	.30	.54	2.16	5.00	120.00	240.00	.45	1.60	62.9	54.9	60.0	51.5
Los Angeles	55.00	.282†	.508†	2.034†	5.085†	115.50	192.50	1.00	2.00	59.6	51.6	57.0	48.6
Minneapolis	57.20	.24†	.432†	1.728†	4.60†	.....	.....	.70	1.80	64.2	56.2	60.7	52.2
Montreal	60.00	.55	.90	3.68	5.95*e*	135.00‡	260.00‡	.48	1.92	.....	.....	.....	.....
New Orleans	51.38	.26	.48	1.69	.....	150.00	220.00	.37	2.10	62.4	54.4	58.8	50.3
New York	52.20	.32	.63	2.52	6.45	.....	.....	.50	1.20	64.9	56.8	62.4	53.9
Philadelphia	50.90	.....	.....	.....	.....	.....	.....	.54	1.80	65.3	57.3	62.8	54.3
Pittsburgh	55.60	.245†	.4725†	1.845†	5.4325	97.80	154.20	.35*g*	2.00*g*	68.5*i*	60.5*i*	66.0*i*	57.5*i*
St. Louis	52.48	.24	.432	1.728	3.96	85.00	170.00	.75*j*	1.60*k*	65.1	57.1	62.0	53.5
San Francisco	55.00	.3275*d*	.5895*d*	2.358	5.895	107.25	178.75	.45	2.50	.....	.....	.....	.....
Seattle	55.35	.35	.63	2.52	6.30	72.00	175.00	.45*f*	1.40*f*	.....	.....	.....	.....

‡ Delivered. † F.o.b. *a* B. & S. class B and heavier, C/L lots, 200 tons and over Burlington, N. J. (base) $49.00. Gas pipe and class A, $3 per ton additional, 4 in., $3 per ton additional, 30 in. and larger usually $2 per ton less. *b* Double strength. *c* List to dealer. *d* List. *e* 30-inch. *f* Less 5% for cash. *g* Culvert pipe. *h* Discounts from standard list for consumers carload prices, except Pittsburgh prices are f.o.b. mill. Base price $200 per net ton. List prices per ft.: ½ in., 8½c.; ¾ in., 11½c.; 1 in., 17c.; 2 in., 37c.; 2½ in., 58½c.; 3 in., 76½c.; 4 in., $1.09; 6 in., $1.92. *i* Applies also at Lorain, Ohio, mills. Chicago delivered base is 2½ points less on butt, 1½ on lap. Freight is figured from Pittsburgh, Lorain, O., Chicago Dist. Billing is from point producing lowest price at destination. WROUGHT IRON PIPE: Base price and list prices per ft. same as wrought steel pipe. Discount for Pittsburgh base: Butt-weld—1 in. and 1¼ in. black 34, galv. 19; 1½ in. black 38, galv. 21½; 2 in. black 37½, galv. 21. Lapweld—2½ in. to 3½ in. black 31½, galv. 17½ in.; 4½ in. to 8 in. black, 32½ galv. 20. *j* Reinforced; spec. C 76-37. *k* Reinforced; spec. C 75-37.

(Continued on page 82)


On the job...
near your job
Bethlehem Pipe
Wherever your newest contract may be located, whether it's in the high Sierras or tidewater Carolina, there's a distributor for Bethlehem Pipe conveniently near you.
And whatever your pipe requirements may be, you'll find your local distributor for Bethlehem Pipe ready to supply you promptly with a few feet or a carload.
BETHLEHEM STEEL COMPANY
BETHLEHEM STEEL

LUMBER, TIMBER, PLYWOOD—PER M FT. B.M., CARLOAD LOTS F.O.B.

	SHORT LEAF YELLOW PINE AND DOUGLAS FIR									LONG LEAF Y. P.	PLYWOOD		
	All S. L. Y. P. is No. 2 common or better and for No. 1 N. C. Box. (Prices in Bold Face)									Merchantable grade up to 20 ft.	Rail freight increment (See note for base price)		
	All Fir planks No. 2 common; Fir timber is No. 1 common. Lengths up to 20 ft. (Prices in italics)												
	1x6, [a][g]	1x8, [a][g]	2x4, [a][g]	2x6, [a][g]	2x8, [a][g]	2x10, [a][g]	3x12, [a][h]	6x12, [a][h]	12x12, [a][h]	2x12, [a][h]	12x12, [a][h]	⅜"	¾"
Atlanta	$19.50	$19.50	$19.00	$18.50	$19.50	$21.00	$26.00	.....	.....	$37.50	$50.50	$17.75	$21.60
Baltimore	31.00	32.50	32.00	33.00	33.00	36.00	46.00	.....	.....	68.00	70.00	17.35*g*	21.15*g*
	33.00	*33.00*	*35.00*	*34.00*	*34.00*	*35.00*	*46.00*	*$46.00*	*$46.00*	.....	.....	.....	.....
Birmingham	21.50	21.50	21.50	21.00	22.00	22.50	40.00	39.00	39.00	.....	.....	16.25	19.85
[1]	23.50	23.50	25.50	23.50	24.00	28.00	55.00	.....	.....	60.00	60.00	.....	.....
Boston	30.00[3]	30.00[3]	32.00[3]	32.00	53.00[1]	60.00[1]	77.00[4]	.....	.....	77.00	77.00	17.35*g*	21.15*g*
	28.00	*28.00*	*32.00*	*32.00*	*38.00*	*38.00*	*45.00*	*45.00*	*45.00*	.....	.....	.....	.....
Chicago	31.00	32.00	30.00	30.00	30.00	32.00	50.00	.....	.....	80.00	90.00	13.80	16.80
	36.00	*37.00*	*37.00*	*37.00*	*38.00*	*39.00*	*48.00*	*48.00*	*48.00*	.....	.....	.....	.....
Cincinnati	31.50	33.00	30.50	31.00	32.00	35.00	44.50	.....	.....	53.00	56.00	16.25	19.85
	.....	.....	.....	.....	.....	.....	*46.00*	*46.00*	*46.00*	.....	.....	.....	.....
Cleveland	34.00	34.50	35.00	35.00	38.00	40.00	46.00	47.50	47.50	80.00	80.00	16.70	20.40
Dallas	28.50	32.50	31.50	29.00	30.50	32.50	52.50	52.50	52.50	63.50	71.00	13.90	16.95
Denver [‡]	45.00[4]	44.00[4]	43.00[4]	41.00[4]	41.00[4]	46.00[4]	48.00[4]	60.00[4]	71.50[4]	.....	.....	9.90	12.05
Detroit	35.00	35.00	35.00	35.00	38.50	42.00	46.00	.....	.....	43.00	58.00	16.25	19.85
	.....	.....	.....	.....	.....	*46.00*	*47.00*	*54.00*	*54.00*	.....	.....	.....	.....
Kansas City	28.00	29.00	29.00	28.00	29.00	30.00	35.00	.....	.....	55.00	60.00	12.25	14.95
	29.00	*29.00*	*30.00*	*28.50*	*29.00*	*30.00*	*35.00*	*39.00*	*39.00*	.....	.....	.....	.....
Los Angeles [a]	*30.00*	*33.00*	*36.00*	*33.00*	*33.00*	*33.00*	*38.00*	*46.00*	*39.00*	.....	.....	9.35*gh*	11.35*gh*
Minneapolis [‡b]	47.00[5]	47.00[4]	.....	.....	.....	.....	.....	.....	.....	.....	.....	12.05	14.70
[‡]	*38.00*	*38.00*	*39.00*	*39.00*	*39.00*	*39.00*	*59.00*	*59.00*	*59.00*	.....	.....	.....	.....
Montreal [7]	*54.00*	*56.00*	*52.00*	*54.00*	*56.00*	42.00	45.00	.....	.....	155.00	165.00	.....	.....
	52.00	*54.00*	*52.75*	*55.75*	*57.75*	*60.75*	*52.00*	*52.00*	*52.00*	.....	.....	.....	.....
New Orleans	24.00	25.00	25.00	24.00	24.00	25.00	46.00	.....	.....	62.00	69.00	13.90	16.95
	29.00	*29.00*	*32.00*	*33.00*	*35.00*	*34.00*	*36.00*	.....	.....	.....	.....	.....	.....
New York [‡]	38.00	39.00	37.00	38.00	38.00	39.00	42.00	.....	.....	80.00*f*	82.50*f*	17.35*g*	21.15*g*
[‡]	.....	.....	*45.00*	*45.00*	*45.00*	*45.00*	*45.00*	*47.50*	*47.50*	.....	.....	.....	.....
Philadelphia	24.00	24.50	25.50	25.50	27.50	27.50	29.00	.....	.....	50.00	57.50	17.35*g*	21.15*g*
	55.00	*56.50*	*39.00*	*38.25*	*39.00*	*39.00*	*44.00*	*38.00*	*38.00*	.....	.....	.....	.....
Pittsburgh	38.10	39.90	47.05	42.00	45.65	46.90	59.70	.....	.....	.....	.....	17.00	20.70
	51.90	*49.00*	*49.60*	*49.00*	*48.90*	*49.25*	*66.55*	*66.55*	*66.55*	.....	.....	.....	.....
St. Louis [‡c]	32.00	33.00	31.00	30.00	31.00	32.00	46.00	*51.00*	*51.00*	73.00	72.00	13.90	16.95
San Francisco [d]	*22.50*	*23.00*	*23.00*	*22.50*	*22.00*	*22.00*	*23.50*	*23.50*	*23.50*	.....	.....	7.30*g*	8.90*g*
Seattle	*21.00*	*21.00*	*21.00*	*21.00*	*21.00*	*21.00*	*26.00*	*26.00*	*26.00*	.....	.....	0.00	0.00

Bold Face type, Southern Pine. *Italics, Douglas Fir.* [1] Long leaf. [2] Roofers' N. C. Pine. [3] Spruce. [4] Native. [5] Western Pine, No. 3 Common. [7] Spruce. [6] Norway Pine. [‡] Delivered. [a] Yard prices. [b] Contractors discount in Minneapolis and St. Paul discontinued May 21, 1938. [c] 5M ft. or less. [d] At ship's tackle. [e] 10% discount taken off. [f] Up to 18 ft.

NOTE: Special concrete form grade Fir Plywood, 4' by 8' panels, 5-ply, sanded 2 sides, water resistant glue, carload lots, delivered per 1,000 sq. ft. surface. Seattle base price on ⅝", $77.00; on ¾", $84.80; price includes oiling and sealing charges. For other centers add rail freight increment from table or proper size. For resin dipped treatment, add $10.50 per M. [g] Lower ate by water shipment. [h] 50,000 lb. minimum.

GLASS, EXPLOSIVES, CHEMICALS

	WINDOW GLASS Discounts from jobbers list, Aug. 15, 1938 Single or Double Thickness		EXPLOSIVES Per lb. 40% Ammonia Gelatin in 50-lb. cases delivered in 200 lb. lots[e]	
	A quality	B quality		
Atlanta	75%	75%	$0.15	
Baltimore	83%	82%	.15	
Birmingham	75%	75%	.105	
Boston	82%	82%	.15	
Chicago	79%	79%	.15	
Cincinnati	79%	79%	.15	
Cleveland	79–10–10%	79–10–10%	.19	
Dallas	78%	78%	.165	
Denver	76–10%	76–10%	.15*c*	
Detroit	79–10%	79–10%	.15	
Kansas City	77–10%	77–10%	.155	
Los Angeles	86%*d*	88%*d*	.1575	
Minneapolis	76%	76%	.155	
Montreal	60–5%*ab*		.17†	
New Orleans	70%	75%	.16	
New York	82%	83%	.20*f*	
Philadelphia	82–10%*d*	83–10%*d*	.15	
Pittsburgh	79%	79%	.1425	
St. Louis	79–10%	79–10%	.155	
San Francisco	78%*d*	83%*d*	.155	
Seattle	85%*d*	88%*d*	.1575	

Price of 60% Ammonia Gelatin is $0.015 per lb. higher than 40%

[a] Disc. from list Sept. 1939. [b] Also less 6% tax exemption. [d] Discount from jobbers' list Sept. 15, 1928.

[e] Urban prices influenced by service charges or local storage and del very regulations, do not consistently reflect quantity prices in less congested areas. [c] F.o.b. Louviers, Colo. [f] In boroughs of Kings, Queens and Richmond, and in Manhattan south of Canal Street, add delivery charge of $6.00 per trip. † F.o.b.

40% Ammonia Gelatin price ranges in other than urban areas, per lb. (except Seismograph Grades)

	C/L 20,000 lb. net	Tons	200 lb. lots
E. of the Miss., except Fla.	$0.105	$0.14	$0.16
W. of Miss., to Rocky Mtn. States, and Fla.	.11 –.12	.135 –.15	.155 –.17
Rocky Mtn. States	.11 –.1325	.13 –.145**	.15 –.165**
Pacific N. W. States	.1075–.115	.1375–.15	.1575–.17
Pacific S. W. States	.1050–.1175	.135 –.1475	.155 –.1625

** F.o.b. Louviers, Colo., or Butte, Mont.

PILES, TIES—F.O.B.

PILES

Prices per linear foot, pine, with bark on, f.o.b. New York; delivered from barge 1½ to 2c. per ft. additional:

Dimensions	Points	Length	Short Leaf Barge	Short Leaf Rail
12 in. at butt	6 in.	30 to 50 ft.	$0.18	$0.195
12 in.—2 ft. from butt	6 in.	50 to 59 ft.	.225	.24
12 in.—2 ft. from butt	6 in.	60 to 69 ft.	.23	.255
14 in.—2 ft. from butt	6 in.	50 to 69 ft.	.24	.265
14 in.—2 ft. from butt	6 in.	70 to 79 ft.	.28	.275
14 in.—2 ft. from butt	5 in.	80 to 85 ft.	.40	.415
14 in.—2 ft. from butt	5 in.	85 to 89 ft.	.42	.482

RAILWAY TIES

Prices f.o.b., per tie for carload lots:

		6' x 8" x 8' Untr.	6' x 8" x 8' Tr.	7' x 9" x 8'6" Untr.	7' x 9" x 8'6" Tr.
Boston	Oak	$1.20	....	$1.65	....
	Pine	1.50	$2.25*a*	2.25	$3.20*a*
New York	S. L. Sap Pine	1.10/1.20	1.85	1.50	2.35
	Mixed Oak	1.45	1.95	1.90	2.45
Birmingham	White Oak	.85	1.50*a*	1.35	1.70*a*
	Southern Pine	.55*b*	1.10*ab*	.70	1.30*a*
Chicago	Oak	....	1.65*a*	....	2.25*a*
	Southern Pine	....	1.53*a*	....	2.13*a*
Los Angeles	Douglas Fir	.64	1.38*a*	.90	1.80*a*
Philadelphia	Red Oak	1.10	....	....	2.40*a*
St. Louis	White Oak	1.12	....	1.50	....
	Red Oak	1.12	1.60*a*	1.50	2.05*a*
	Sap Pine or Cypress	1.02	....	1.40	....
San Francisco	Douglas Fir	.90*bcf*	....	....	1.95*caf*
Montreal	Birch or Maple	.65	1.05*a*	.75	1.35*a*

Tr.—Treated; Untr.—Untreated. [a] Creosoted. [b] 6" x 8" x 8'6". [c] Empty cell. [d] Zinc. [e] Green. [f] At ships tackle.

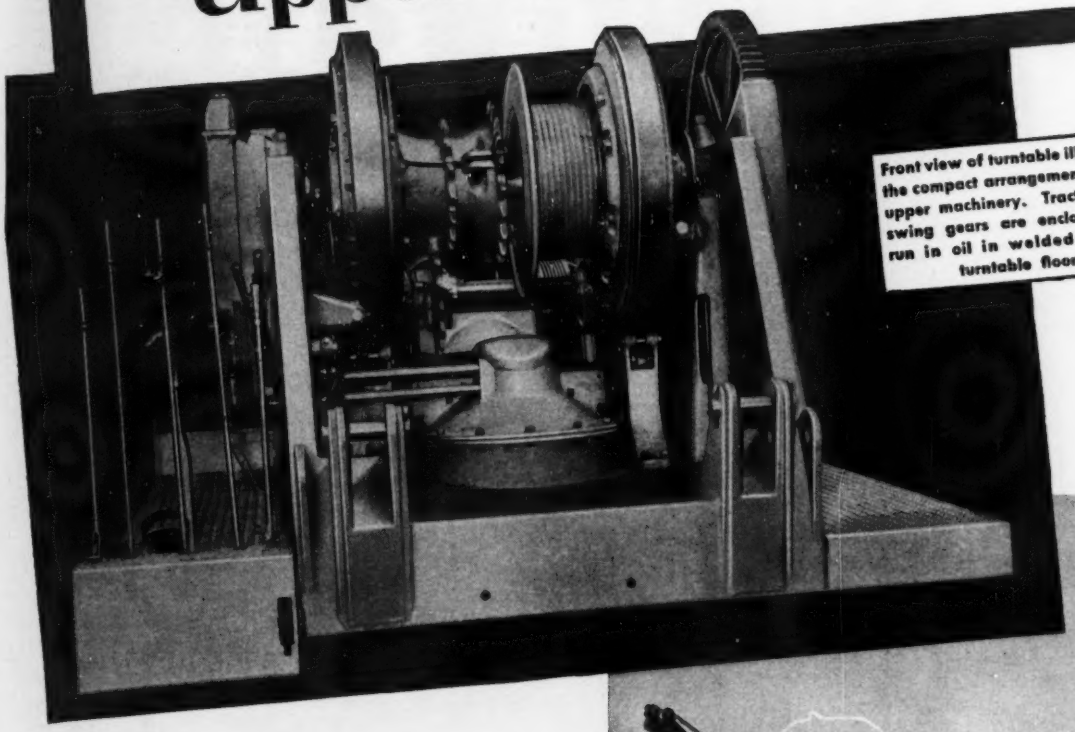
CHEMICALS

Water, sewage treatment, road work, f.o.b. carlots, New York

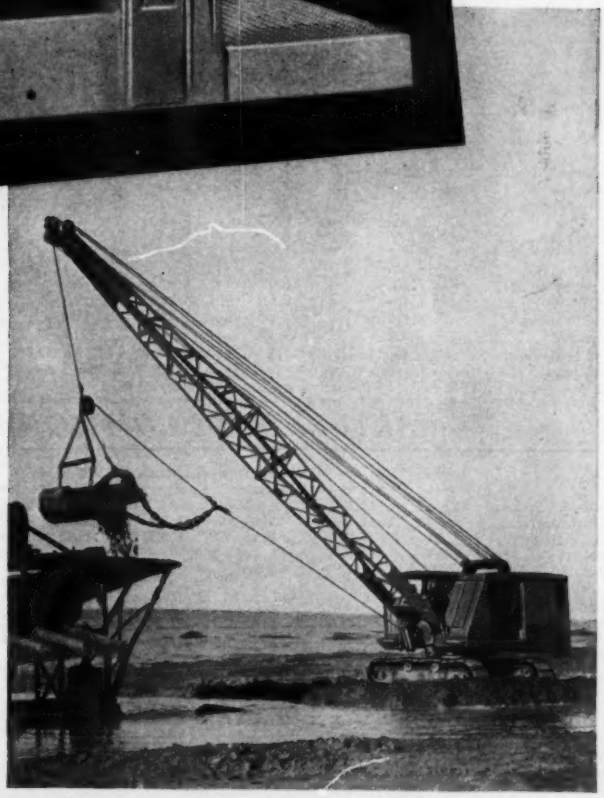
Bleaching powder, in drums, f.o.b. works, per 100-lb	$2.00–2.85
Chlorine cylinders, liquid, per lb. delivered	.0520
Calcium chloride, 77–80%, flaked, in 400-lb. drums or 100-lb. moisture proof bags, delivered, per ton	20.50–35.50
Silicate of soda, 52 deg., in drums, f.o.b. works, per 100 lb	1.40
Soda ash, 58%, in paper bags, per 100 lb. dense	1.10
Sulphate of aluminum, commercial, in 100-lb. bags, per ton	23.00
Sulphate of copper, in bbl., per 100-lb	4.60

(Continued on page 84)

Simple and Compact Upper Machinery



Front view of turntable illustrates the compact arrangement of the upper machinery. Traction and swing gears are enclosed and run in oil in welded case on turntable floor.



Simplicity Reduces Repair Bills

Profit-earning depends on high-speed production and *low repair bills.* Simple design and construction of the most important machinery . . . the upper machinery . . . on Koehring 604 Excavators, keeps repair costs low. Enclosed gears running in oil, anti-friction bearings and small number of operating parts . . . easily accessible for operating adjustment . . . are important reasons why Koehring Excavator maintenance costs do not wipe out the job-profit.

KOEHRING COMPANY
MILWAUKEE, WIS.

KOEHRING K HEAVY DUTY

HEAVY-DUTY CONSTRUCTION EQUIPMENT

STRUCTURAL CLAY BUILDING TILE, BRICK AND LIME—DELIVERED

	STRUCTURAL CLAY TILE — PARTITION — SCORED Per M. lots of 2,000 pieces or over, 3x12x12 in.	4x12x12 in.	8x12x12 in.	STRUCTURAL CLAY TILE — LOAD BEARING — SCORED Per M. lots of 2,000 pieces or over, 8x12x12 in.	10x12x12 in.	12x12x12 in.	BRICK Per M. in quantity. Common backing	Straight hard	LIME Per ton, Hydrated finishing	In paper Common hydrated	Carload lot Pulverized or lump
Atlanta	$90.00	$100.00	$185.00	$220.00	$230.00	$240.00	$12.50	$16.00	$26.50	$17.00	$17.00
Baltimore	74.00	80.00	165.00	190.00	235.00	290.00	13.00*k*	16.00*k*	18.50	14.50	2.50*m*
Birmingham	80.00	85.00	160.00	191.00	252.00	288.00	13.50	16.50	17.11	11.64	1.55*lo*
Boston	95.05*a*	101.40*a*	190.05*a*	215.35	279.20*a*	319.10*a*	17.00*i*	20.00*i*	20.00	13.00	17.00*l*
Chicago	68.10	72.60	135.90	185.70	234.90	268.10	11.00	12.00	16.40	14.40	12.20
Cincinnati	61.40	65.30	121.90	141.50	187.30	215.20	16.00	16.00	15.90	13.00	.....
Cleveland	56.50	60.00	112.50	135.00	182.50	209.00	16.00	16.00	15.00*j*	12.00*j*	2.75*j/m*
Dallas	62.50	67.00	135.00	162.00	177.00	263.00	11.50*i*	13.50*i*	19.00	15.00	.95*n*
Denver	84.80	95.40	152.70	178.20	212.00	227.90	13.50	17.00	28.00*j*	17.00*j*	35.00*j*
Detroit	68.90	73.50	137.80	175.50	232.00	265.00*e*	15.70	17.00	13.50	11.75	12.75
Kansas City	64.50	66.50	86.00*e*	125.50	159.75	186.25	14.00	24.00	19.10	12.80	10.70
Los Angeles	73.50†*r*	89.50†*r*	128.00*c*†*r*	210.00†*r*	368.00†*r*	460.00†*r*	13.50	15.00	16.50	.....	17.50
Minneapolis	66.70	71.20	122.50	141.30	149.30	209.40	13.10	15.10*g*	21.00	16.00	19.00
Montreal	72.20*a*	77.50*a*	142.50*a*	76.00*a*‡	165.80*a*	248.70	20.75	28.00	23.50	17.00	12.00
New Orleans	72.00	72.00	135.00	162.00	210.00	260.00	14.00	.....	18.30	12.08	14.10
New York	87.00	93.00	128.00*c*	193.30*h*	238.80*h*	295.60*h*	13.00	.....	19.00	14.00	17.50
Philadelphia	80.50	85.90	161.10	193.30	238.80	295.60	15.00*s*	18.00*s*	15.50*g*	9.55*g*	12.05*q*
Pittsburgh	58.40	62.40	116.85	124.00	153.00	193.75	17.00	25.00	17.90	15.40	2.55*m*
St. Louis	62.00	88.00	120.00	180.00	234.00	280.00	16.00	17.00	22.00	16.00	2.30*lm*
San Francisco	84.00	94.50	225.00	.....	.....	.....	14.00	17.00	.55*e*	.40*e*	2.05*m*
Seattle	100.00	110.00	220.00	.....	.....	.....	17.50*f*	17.50*f*	20.00*s*	20.00*s*	20.00*s*

† F.o.b. *a* Smooth. *b* Carload lots delivered to job. *c* 6 x 12 x 12 in. *d* Not load bearing. *e* 48 lb. tile. *f* Less $1.00, ‡ cash 15 days, balance 30 days. *g* Selected common. *h* F.o.b. Perth Amboy, N. J. *i* 50c per M. off for cash. *j* LCL. *k* $1.00 discount if paid in 10 days. *l* Lump. *m* Per bbl., 180 lb. *n* Per sack 100 lb. *o* Per bbl., 200 lb. *p* 280 lb. bag. *q* 25-ton cars. *r* 5% discount 10 days. *s* 2% off cash. *t* 5½ x 8 x 12.

PAINT, ROOFING—F.O.B. CARLOAD LOTS

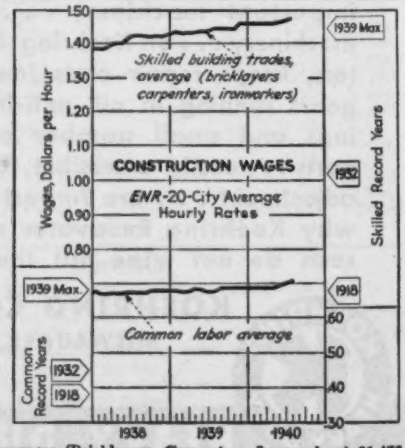
	RED LEAD 600-lb. Per 100 lb. in. (Approx.) Dry *a*	WHITE LEAD bbl. Per 100 lb. in oil	READY-MIXED PAINT Per gal., drums Graphite *b*	Aluminum *c*	Ferric Oxide *d*	ROOFING SUPPLIES Carload lots f.o.b. factory Rolls, slate surfaced, 85–90 lb., per sq.	Asphalt felt, per 100 lb.	Tar felt, per 100 lb.	Asphalt coating per gal.	Tar pitch 350 lb. bbl. per ton
Atlanta	$8.75	$13.125	$1.30	$2.25	$1.20	$1.90	$1.61	$1.61	$0.27	$29.00
Baltimore	8.50	12.75	1.70	2.45	1.95	2.20	2.75	2.75	.45	27.50
Birmingham	8.75	13.125	2.35	2.75	1.85	1.98	1.57	1.57	.32	27.00
Boston	8.50	12.75	1.75‡	2.30‡	1.75‡	1.75‡	1.85*h*‡	1.85*h*‡	.50‡	20.00‡
Chicago	8.50	12.75	1.90	2.63	1.68	1.48*h*/*f*	1.55*h*/*f*	1.55*h*/*f*	.26/*f*	21.00/*f*
Cincinnati	8.50	12.75	1.65	2.85	1.55	2.40	2.30*h*	2.30*h*	.36	21.00
Cleveland	8.50	12.75	1.90	2.60	1.60	1.75	1.50*h*	1.50*h*	.35	21.00
Dallas	8.50	13.50	1.88	2.11	....	2.75*h*	2.60*h*	2.70*h*	.275	34.00
Denver	9.00	13.25	1.65	2.40	.171	2.45	4.00	3.00	.35	18.00
Detroit	8.50	12.75	2.00	3.00	1.75	2.40	2.80	2.50	.35	26.00
Kansas City	8.50	12.875	2.15*g*	2.40*g*	1.67*g*	1.93	2.09	2.09	.25	25.00
Los Angeles	8.75	13.25	1.60	2.25	2.30	2.10	1.80	....	.45	....
Minneapolis	8.50	12.875	....	....	....	1.75*i*	1.50*h*/*i*	1.50*h*/*i*	.29	21.00
Montreal	9.25	9.00	2.25	2.80	2.35	2.53	1.68	1.78	.70	1.68*h*
New York	8.50	12.75	1.88	2.55	1.60	1.75	1.50*h*	1.50*h*	.23	24.00
Philadelphia	8.50	12.75	....	....	....	1.92*e*	1.99*h*/*e*	1.99*h*/*e*	.41*e*	21.00
Pittsburgh	8.50	12.75	1.05	1.95	.80	1.95	1.94*h*	1.94*h*	.34	26.00
St. Louis	8.50	12.75	1.90/2.00	2.60/2.85	1.60/2.00	1.45	1.45	1.45	.22	21.00
San Francisco	8.75	13.00	1.80	2.75	1.10/2.00	1.65	2.26	2.26	.28	17.00
Seattle	8.75	13.25	1.80	2.60	....	1.81	1.30/2.00	1.75	.35	28.00/*f*

‡ Delivered. Note: Red lead in oil 50c. higher than white lead in oil. Red lead prices change frequently due to pig lead price changes. *a* *b* U. S. War Dept. Spec. 3–49A. *c* ASTM Spec. D266–31. *d* 80% minimum ferric oxide. *e* Subject to 25% discount. *f* Distributors' price to contractors. *g* 5 gal can. *h* Per roll, 65 lb. *i* Minneapolis and vicinity. *j* Asphalt pitch. *k* per 100 lb. *l* per lb.

SKILLED AND COMMON WAGE RATES—PER HOUR

	Bricklayers	Carpenters	Struct. Iron Workers	Hoisting Engineers	Plasterers	Common Labor Building	Common Labor Heavy Const.
Atlanta	$1.25	$1.00	$1.375	$1.00/1.25	$1.25	$0.40/.50	$0.40/.50
Baltimore	1.50	1.25	1.65	1.50	1.375	.5625	.5625
Birmingham	1.50	1.125	1.25	1.125	1.00	.30/.40*a*	.30/.40*a*
Boston	1.50	1.375	1.625	1.375/1.625	1.625	.60/.85	.60/.85
Chicago	1.70	1.625	1.70	1.70	1.70	1.025	1.025
Cincinnati	1.625	1.45	1.55	1.375/1.525	1.625	.75	.75
Cleveland	1.625	1.375	1.50	1.50/1.75	1.50	.90	.90
Dallas	1.50	1.00	1.25	1.00	1.50	.40/.50	.40/.50
Denver	1.65*b*	1.43*b*	1.43*b*	1.43	1.50*b*	.7185	.7185
Detroit	1.50	1.25	1.625	1.50	1.55	.60/.75	.60/.775
Kansas City	1.625	1.375	1.50	1.50	1.575	.85	.50
Los Angeles	1.25	1.10	1.375	1.375	1.50	.625	.625
Minneapolis	1.375	1.25*d*	1.50	1.40	1.50*e*	.85	.85
New Orleans	1.50	1.25	1.375	1.375	1.40	.40	.55
New York	2.00*b*	1.85*b*	2.00	2.00*b*	2.00*e*	1.031	.875/.95
Philadelphia	1.65*b*	1.40	1.65	1.50	1.75	.70	.70
Pittsburgh	1.75	1.50	1.50	1.25	1.75	.80	.70
St. Louis	1.50	1.50	1.75	1.50/2.00	1.75	.45/.875	.875
San Francisco	1.75*c*	1.25	1.60	1.50	1.60*c*	.813	.813
Seattle	1.60*c*	1.25	1.50*c*	1.50*c*	1.60*c*	.90*c*	.90*c*
Montreal	.80	.70	.75	.65	.80	.45	.40

a PWA. *b* 7 hr. day. *c* 6 hr. day. *d* 35 hr wk. *e* 30 hr. wk.



ENR Skilled Average: (Bricklayers, Carpenters, Ironworkers) $1.476
ENR Common Average: $0.707